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Storm Water Problems and Control in Sanitary Sewers

OAKLAND AND BERKELEY, CALIFORNIA

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STORM WATER PROBLEMS AND CONTROL IN SANITARY SEWERS

Oakland and Berkeley, California

by

Metcalf & Eddy, Inc.
Engineers
Boston - New York - Palo Alto

for the

ENVIRONMENTAL PROTECTION AGENCY
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ABSTRACT

An engineering investigation was conducted on storm water infiltration into sanitary sewers and associated problems in the East Bay Municipal Utility District, Special District No. 1, with assistance from the cities of Oakland and Berkeley, California.

Rainfall and sewer flow data were obtained in selected study subareas that characterized the land use patterns predominant in the study area. Results obtained were extrapolated over larger drainage areas. A computerized flow routing program for the sewer system was used in this analysis.

Ratios of infiltration to rainfall in the study subareas range from 0.01 to 0.14. Ratios of peak wet weather flow to average dry weather flow range from 2.1 to 9.1. About 11.1 percent of the rainfall enters the sanitary sewer system; 30.6 percent of the infiltration is contributed by the 4 percent of the study area that has combined sewers.

Problems associated with infiltration and resulting overflows and bypasses are: 1) pollution of San Francisco Bay, 2) operational difficulties at the treatment plant, and 3) danger to public health, property damage, and nuisance.

Estimated costs for the most feasible combinations of solutions to these problems, consisting of treatment plant improvements, separation of remaining combined sewers, partial treatment of overflows, and sewer improvements, range from approximately $42 million to $94 million. Specific recommendations for subsequent developmental programs are presented; complete implementation of the recommended plan will take about 7 years.

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Pursuant to Executive Reorganization Plan No. 3 of 1970, effective December 2, 1970, and Environmental Protection Agency Orders Nos. 1110.1 and 1110.2, all references to the Federal Water Quality Administration herein shall be to the Environmental Protection Agency, Water Quality Office.

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SECTION 1

CONCLUSIONS AND RECOMMENDATIONS

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SECTION 1

CONCLUSIONS AND RECOMMENDATIONS

This engineering investigation of the sanitary sewer systems and water pollution control facilities of the East Bay Municipal Utility District, Special District No. 1 was conducted by Metcalf & Eddy, Inc., with the assistance of the City of Oakland, the City of Berkeley, and the East Bay Municipal Utility District in the data collection phase, under the terms of Contract 14-12-407. The study was directed toward determining the magnitude and sources of storm water infiltration and the nature of problems associated with the resulting overflowing of sewers and bypassing at the water pollution control plant. Alternative solutions for controlling these problems were defined, and preliminary cost estimates were made.

The following conclusions are based on the data collected and evaluated during this investigation and presented in the body of the report.

CONCLUSIONS

1. Infiltration ratios (the volume of infiltration to the volume of rainfall) were defined both for gross infiltration and for infiltration from each of two sources: percolation and direct connections. The gross infiltration ratios for selected study subareas ranged from 0.01 to 0.14, depending upon land use, topography, and age or condition of the sewer system. The drainage area that contributes directly to Pump Station A was found to have a ratio of 0.246. About 60 percent of the infiltration was attributed to percolation and 40 percent to direct connections for the eight subareas evaluated.

2. Variations in waste water characteristics during storm conditions were not well defined, but general trends were detected. Practically all of the parameters showed a sharp increase in concentration as the flow rates in the sewer increased. These concentrations returned to normal, or below normal, by the time the peak flow rate had reached the sampling station or slightly thereafter. The exception was the ratio of suspended solids to total solids. This ratio seemed to increase gradually so long as substantial quantities of extraneous flow continued.

3. The average concentrations of pollutants representing the quality of overflows and bypasses were:

Location	BOD, mg/L	Grease, mg/L	Coliform bacteria, MPN/100 ml
South interceptor	370	105	3.7×10^8
North interceptor	105	21	1.5×10^7
Water pollution control plant	135	40	--

4. An analysis of geographic rainfall distribution was made, from which a correlation was found to exist between rainfall at various locations (total per storm) and that at the Metropolitan Oakland International Airport. Relationships were found for storms having general rainfall distribution, ranging from factors of 0.9 to 1.55 times the rainfall at the Oakland Airport. No relationship could be defined for temporal distribution of rainfall, nor could a pattern be defined for rainfall during the day.

5. A statistical analysis of rainfall data taken at the Oakland Airport indicated that the maximum storm event for the rainy season under investigation, 1968-69, had a return frequency of about 1.9 years.

6. From an analysis of the first four hours of each of the largest storms during the season together with recorded overflow occurrences at two pumping stations and bypasses at the water pollution control plant, both a "marginal overflow storm" and a "marginal bypass storm" were defined. The storm conditions were defined by the cumulative rainfall and the time of rainfall as shown on Figures 24 and 25 on page 93. These conditions indicate the intensity and duration of a storm that is needed to cause overflows and bypasses in the East Bay Area.

7. The ratios of peak wet weather flow, measured in the sewers, to average dry weather flow ranged from 2.1 to 9.1. Ratios were computed for both average daily flow and for average flow for the same time of day; these are presented in Table 11.

8. The relative magnitude of the major sources of storm water infiltration for the study area was concluded to be as follows:

   a. The equivalent storm water infiltration ratio for the East Bay Area is 0.111, i.e., 11.1 percent of the total volume of rain falling on the area enters the sanitary sewer system.

   b. Approximately 30.6 percent of the total volume of infiltration is contributed by infiltration and runoff from those areas which have combined sewers, composing 4 percent of the study area.

c. Approximately 3.7 percent of the total volume of infiltration is contributed by the Pump Station A drainage area, composing 1.4 percent of the study area.

d. In the remaining 94.6 percent of the study area, approximately 26.2 percent of the total infiltration is contributed via direct connections, and 39.5 percent is contributed via percolation or pipe leakage.

9. During an average year (a year having an average rainfall), the following conditions were estimated to occur:

a. The total rainfall is 17.57 inches at the Oakland Airport.

b. Bypasses at the water pollution control plant occur an average of 11 to 12 times per year, and it was concluded that sewer overflows occur the same number of times, based on similarities between the storms causing each condition.

c. Approximately 1,660 MG of extraneous flow (6.8 percent of the rainfall) reach the water pollution control plant and 1,040 MG of extraneous flow (4.3 percent of the rainfall) overflow before reaching the plant.

d. Bypassing at the plant occurs for 110 hours at an average rate of 168 mgd, resulting in a discharge of 780 MG of waste water that consists of 480 MG of extraneous flow and 300 MG of base or dry weather flow.

e. Approximately 1,180 MG (1,660 - 480) of extraneous flow reaching the plant are treated before discharge.

f. Overflow volumes were estimated to equal 90.5 MG per event, with 62.5 MG occurring south and 28.0 MG occurring north of the plant.

10. Four types of overflows or bypasses were found and are listed below. The first three contribute to the discharge of untreated combined sewage to the Bay; the fourth involves the overflowing of sewage within the system resulting in nuisance, property damage, and public health hazards.

a. Intersystem overflows - occurring at cross-connections between storm drains and sanitary sewers, resulting at times in the discharge of raw sewage to the Bay.

b. System overflows - occurring at specially designed structures near the interceptor sewer to discharge excess flow to the Bay.

c. Water pollution control plant bypass - manually controlled bypass around the sedimentation basins to prevent damage to the plant's mechanical equipment.

d. Intrasystem overflows - occurring at manholes or other openings and broken sewers.

11. Two operation and maintenance problems were found at the water pollution control plant which result from storm water infiltration.

a. During wet weather large quantities of fine sand and silt, which would cause mechanical failure if the flow were not bypassed, settle in the sedimentation basins. The additional sand and silt that settles before bypassing begins is pumped to the digester, hence taking up digester volume and causing unnecessary cleaning problems.

b. During periods of bypassing, large volumes of grease accumulate in the outfall sewer and in the outfall transition structure. Grease that clings to the wall of the outfall later breaks loose in sizeable chunks and contributes to the visible, floating grease. Grease that accumulates in the structure requires substantial maintenance effort for cleaning.

12. Based upon a review of the various water quality objectives that would be applicable to the waters of Central San Francisco Bay, the following water quality objectives were selected for evaluating the effects of overflows and bypasses:

Parameter	Unit	Objective
a. Dissolved oxygen	mg/L	Minimum of 5.0
b. Floatable material		
On water surfaces	g/sq m	Maximum of 0.02
On shore areas	g/sq m	Maximum of 0.3
c. Total coliform bacteria	MPN/100 ml	Not more than 20% of samples exceeding 1,000.

13. An investigation of the background concentrations of the selected water quality objectives produced the following conclusions:

a. Average dissolved oxygen levels in the near-shore waters are well above the minimum objective of 5.0 mg/L during the rainy season. The levels of DO that fall below the minimum objective during the months of January and February were concluded to result from a combination of storm sewer discharges and sanitary sewer overflows.

b. Using the guide for floatable grease emissions as set forth in the Bay-Delta study, the dry weather grease emissions were found to be acceptable. This guide would permit 310 pounds per day of floatable grease in Water Quality Zone 3 and 375 pounds per day in Water Quality Zone 4.

c. With the exception of the Bay side of Alameda, the shoreline waters of the East Bay Area were found to exceed the allowable concentrations of coliform bacteria. The offshore waters from Albany, Berkeley, and Emeryville were found to meet the objectives normally, but to exceed them periodically, probably because of a combination of sanitary sewer overflows and storm sewer discharges.

14. The evaluation of the effects of overflows and bypasses on the selected water quality objectives in the near-shore waters indicated that only the objectives for floatable materials and coliform bacteria were exceeded. Except for the possibility of localized problems, the dissolved oxygen content was not appreciably affected. During and following overflowing and bypassing, appreciable concentrations of floatable material (grease) were estimated to persist for six days in Water Quality Zone 3 and for two days in Water Quality Zone 4. The mechanisms of floatable grease removal were dispersion and oxidation. Coliform bacteria were estimated to return to within 10 MPN per 100 ml of the normal background concentrations within 2.6 days in Water Quality Zone 3 and 1.5 days in Water Quality Zone 4. Both dispersion and bacterial die-away were mechanisms in reducing bacterial concentrations.

15. It was concluded that no single action, short of total sewer replacement, could be found to solve the infiltration problems. Therefore, a series of component solutions was evaluated that, when combined, would represent an alternative solution. These component solutions consisted of various improvements to the sewer system, addition of holding basins or overflow treating facilities, additions and revisions to the water pollution control plant, and, where applicable, suggested improvements in sewer design and construction practices. Estimated costs for each of four alternatives ranged from approximately $42,000,000 to $94,000,000.

16. The most practical and the lowest cost combination of component solutions was concluded to be Alternative No. 3, consisting of the following steps: 1) complete about 50 percent of the remaining sewer separation program, 2) provide improvements to the water pollution control plant, 3) locate and disconnect catchbasins that are presently connected to the sanitary sewer system, 4) provide partial treatment for the remaining overflows, and 5) eliminate sewer system bottlenecks.

17. The time period needed to implement the plan of Alternative No. 3 completely was estimated at seven years.

RECOMMENDATIONS

On the basis of the preceding conclusions, the following recommendations are presented:

1. That Alternative No. 3 of the solutions summarized in Table 30 be adopted as a plan for solving the problems of storm water infiltration and overflows. Specific recommendations are presented for the component solutions as follows:

   a. EBMUD, Special District No. 1 should initiate detailed studies, including pilot plant testing if necessary, both for the proposed improvements to the plant and for new overflow treatment facilities.

   b. Each of the six East Bay Area cities should initiate studies and surveys to locate the remaining combined sewers, catchbasins attached to sanitary sewers, and bottlenecks in the sewer system.

   c. Each of the participants should undertake final design and construction of its respective improvements as soon as funds can be made available.

   d. The first phase of construction should include only one overflow treatment facility. After this facility has been evaluated and proven, the remaining treatment facilities should be designed and constructed.

2. That a study of the possible application of electrostatic disinfection equipment for use at overflow treatment facilities be sponsored by EBMUD, Special District No. 1 concurrently with other pilot plant testing or studies.

3. That the City of Oakland sponsor a specific study to determine the application and limitations of the addition of polymers for increasing the effective sewer capacity. The recommended site for application of this process is the Fifth Street trunk sewer, along the south shore of Lake Merritt. Improvement of the flow capacity of the sewer would help

relieve the problems during periods of wet weather flows, thereby minimizing surcharging and overflowing at manholes along the sewer. This procedure would serve as an interim measure until a more permanent solution could be achieved. This recommendation would only be applicable after a substantial improvement in hydraulic capacity of the interceptor is made.

4. That one of the East Bay Area cities sponsor a study to evaluate the use of cure-in-place, plastic pipe for lining sewers. The study would determine the costs and techniques of installation of a glass-fiber reinforced plastic liner pipe that could possibly substitute for conventional sewer replacement programs, producing equivalent results but at a lower cost.

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SECTION 2

INTRODUCTION

THE PROBLEM AND THE STUDY AREA

Infiltration into sanitary sewers has plagued engineers and municipal officials for many years. Every sewer system is subject to infiltration to some degree, depending upon the condition of the sewers, the level of groundwater, and the soil conditions. The extraneous flow resulting from infiltration into sewers uses up valuable hydraulic capacity and subsequently reduces the treatment capability of water pollution control facilities by creating abnormally high flow rates that upset biological and normal treatment activities.

Infiltration is usually defined as leakage of water into sewers from the surrounding ground during both dry and wet weather. This study is concerned specifically with infiltration which occurs only during, and as a result of, wet weather, that is, storm water infiltration, regardless of the source of the storm water. Therefore, the term, "infiltration," as used in this report and when not qualified, refers to the storm water extraneous flow which enters sanitary sewers only during wet weather conditions; or, in some places, it refers to the process by which such extraneous water enters the sanitary sewers. Among the sources of extraneous water are the following:

1. Groundwater and vadose water that enters the sanitary sewer system via defects in the sewer.

2. Surface water that enters the sanitary sewer system via direct connections of roof drains, foundation drains, surface inlets, catchbasin leads, or other storm water drains, or via manhole covers or other means.

3. Storm water which is bypassed or overflowed to the sanitary sewer system from combined sewers or storm drains.

It might be said that the quantity of storm water infiltration in a sanitary sewer at any time is computed as the difference between the actual flow at the time and the quantity which would have been flowing if the rainstorm had not occurred.

In connection with the storm water - infiltration problem, it is necessary in some instances to mention the dry weather infiltration, which consists of all infiltration other than storm water infiltration. Where such usage occurs in this report, either it is qualified or the meaning is obvious.

Storm water infiltration results from the use of inadequate construction materials, poor construction practices, and direct connections. Until recently, sewer construction materials and practices were frequently conducive to infiltration because poor joints were produced when the pipe was laid. Rigid jointing materials fractured during pipe settlement, permitting groundwater to enter or sewage to escape the pipe. With improvements in construction materials such as flexible plastic joints, the problem of infiltration in recent sewer construction is greatly reduced. However, sewers that were built before the improvements in construction materials and methods were developed--around 1960--have many years of useful life remaining, and it is not likely that they will be replaced before the end of that useful life is reached. Direct connections, such as roof, yard, and foundation drains, add to the quantity of infiltration, especially during wet weather. In many instances, these connections are not only difficult to locate but also difficult to disconnect because of the political ramifications involved. For these reasons, the general problem of infiltration will continue for many years.

The widespread nature and magnitude of the storm water infiltration problem was illustrated in a 1967 national survey made by the American Public Works Association for the FWPCA, now called the FWQA. Fifty-three percent of the jurisdictions reported wet weather infiltration problems, but only 14 percent reported dry weather infiltration problems. Moreover, 197 jurisdictions indicated that water pollution control plant bypassing occurred for an average of 350 hours per year at each plant during wet weather conditions for the following reasons:

1. The plant was not designed to handle grit and debris carried by storm flow.

2. High river stages did not permit operation of the plant without excessive pumping.

3. The receiving stream flow was of such magnitude that treatment was not deemed necessary by local officials.

4. The plant had insufficient capacity.

Many jurisdictions also reported overflows at locations within the system other than at bypasses around the plant.

Because the study area selected for this investigation reflects many of the problems cited above, it is expected that the results will benefit many cities in addition to those within the study area.

The study area is a portion of the East Bay Area of the San Francisco Bay metropolitan region in California. The East Bay Area normally refers to all of the cities located along the east side of the San Francisco Bay, but, in this report, the term is limited to the six cities that compose the East Bay Municipal Utility District, Special

District No. 1: Oakland, Berkeley, Alameda, Albany, Emeryville, and Piedmont. The City of Oakland, the City of Berkeley, and EBMUD, Special District No. 1 participated in and contributed materially to this study. Therefore, the detailed investigations and resulting data were developed mainly from areas within these two cities and from the water pollution control plant operated by EBMUD, Special District No. 1. This plant, located in Oakland near the east end of the San Francisco-Oakland Bay Bridge, was built in 1948-1951 to serve the entire study area.

HISTORICAL BACKGROUND

Prior to 1951, the East Bay Area cities were served by individual systems of combined sewers which conveyed all waste waters together with storm water runoff from each of the many drainage areas directly into the near-shore waters of the Bay. By 1940, the pollution state of the East Bay waters had reached an intolerable condition.

Realizing that action was necessary, the City of Berkeley sponsored a comprehensive investigation by a board of consulting engineers, of the collection, treatment, and disposal of sewage and industrial wastes from seven East Bay cities (the six cities in EBMUD, Special District No. 1 and Richmond). The results, published in the "1941 Sewage Report" (1), provided the basis for most of the improvements that have been made to date. The four recommendations most pertinent to this study were that:

1. Dry weather sewage flow from the East Bay cities and from all areas naturally tributary thereto be collected for primary treatment and disposal in the vicinity of the San Francisco-Oakland Bay Bridge.

2. The construction and operation of the recommended sewage collection system and water pollution control plant be performed by the East Bay Municipal Utility District.

3. A program be at once developed whereunder existing combined sewage systems be systematically separated into sanitary sewers and storm sewers.

4. A program of repair and reconstruction be inaugurated throughout the East Bay Area aimed at the reduction of the excessive, unjustifiable amounts of infiltration entering the sewers.

By 1951, the first and second recommendations had been put into effect. The third recommendation was also subsequently implemented, and by 1969 approximately 96 percent of the sewer separation program had been completed. The fourth recommendation is the one that has been given the least attention. Thus, after some 30 years, infiltration problems still plague the East Bay Area cities.

Because a major part of the sewer separation program was completed prior to 1960 before the improved jointing methods for vitrified clay sewer pipe were introduced, many of the problems of leakage and infiltration that were previously common to the combined sewer system were built into the new sanitary sewer systems. Moreover, not all of the present sanitary sewer system represents newly constructed sanitary sewers. In many cases, the old combined sewer was converted to a separate sanitary sewer and a new storm sewer was constructed. Therefore, the problems inherent in the combined sewer system were carried into the sanitary sewer system.

Compounding the infiltration problem was the fact that the repair and reconstruction of sewers was not extensively undertaken. Stopgap measures were used wherever problems of inadequate capacity were found. Consequently, the sewer systems of the East Bay Area cities have many cross-connections between storm and sanitary sewers, direct connections into sanitary sewers, parallel relief sewers, and overflows directly to the Bay waters.

Various attempts have been made by EBMUD, Special District No. 1 to measure the extraneous flow resulting from storm water infiltration. Because of the many complications within the system, it was never certain that all of the extraneous flow actually tributary to a particular sewer was measured, nor could all of the problems caused by the variable hydraulic conditions posed by tide elevation, variations in pumping rates, and manual operation of diversion structures be overcome. The conclusions in each case have been that there is a substantial increase in flow rates during rainfall, but no actual numbers could be assigned with any degree of confidence.

In view of the complexity of the sewer system and the unsuccessful previous efforts to define the magnitude of the infiltration problem, it was concluded for this study that a direct method of measuring all extraneous flows during rainfall would be extremely difficult and expensive. No method short of measuring flows in all major storm sewers and sanitary sewers would portray the complete picture.

Water that overflowed within the system from sanitary sewers into storm sewers would not be included in flow measurements taken in sanitary sewers. Also, the differentiation of storm water runoff, storm water infiltration, and sanitary sewage would be extremely difficult. The design and construction characteristics of the connections between the old trunk sewers and the interceptor sewers are not conducive to flow measurement. Limitation of access, interference by tidal variation, and the somewhat unpredictable manual operation of the pumping station and various diversion structures along the interceptor sewer all compound the problem of flow measurement. For these reasons, a different

approach to measurement of extraneous flow resulting from storm water infiltration was necessary for this study.

PURPOSE AND SCOPE OF THE STUDY

The purpose of this investigation was to define both quantitatively and qualitatively the magnitude of storm water infiltration in the sanitary sewer systems of the East Bay Area cities, to define the problems associated with large volumes of infiltration, to ascertain what alternative solutions are available to alleviate the problems, and to provide preliminary designs and cost estimates for the recommended alternative solutions.

The specific objectives paraphrased from the STATEMENT OF WORK of the contract were to:

1. Define the study area and select subareas for intensive study.

2. Collect all existing pertinent data.

3. Measure both dry and wet weather flows in each subarea.

4. Sample both dry and wet weather flows in each subarea.

5. Measure precipitation with sufficient gages to provide a real definition of rainfall characteristics.

6. Establish the characteristics of both dry and wet weather discharges to the receiving waters.

7. Define the background water quality and the water quality objectives for the receiving waters.

8. Define the pollutional effects of overflows and bypasses on the receiving waters.

9. Develop alternative solutions for controlling or eliminating overflows and bypasses from the receiving waters.

10. Prepare design criteria and cost estimates for the alternative solutions.

11. Select and recommend the best alternative solution.

12. Prepare a final project report.

METHOD OF APPROACH

The approach used in this study to accomplish the preceding objectives entailed the following steps:

1. Selection of subareas that typified various conditions prevalent in the study area as a whole, such as sizes and types of sewers; topographic, soil, and groundwater conditions; and land use categories.

2. Characterization of infiltration in each of the subareas and the development of infiltration ratios related to rainfall and land use. Characterization was based on actual field measurements of rainfall and flows in sewers and the sampling and analysis of waste water flows.

3. Development of a mathematical flow routing program which would simulate the dry and wet weather flows in any given drainage area. The routing program was valuable for defining the components of infiltration; determining if sewer system overflows occur and, if so, how much; and evaluating the effect of holding tanks on the outflow hydrograph. The routing program further aided in confirming a method of extrapolating measured or computed volumes of extraneous flow from a small drainage area to the total drainage area.

4. Evaluation of causes and effects of infiltration and the development of alternative solutions to the problem. The evaluation was based on defining the problems caused by infiltration and determining the pollutional effects of overflows and bypassed flows on San Francisco Bay from available data.

5. Development of feasible alternative solutions, cost estimates, and recommendations.

The project report is divided into three volumes. This volume contains a description of the study area and of the data collection program, the development and discussion of the problems and alternative solutions, and the conclusions and recommendations. The "Technical Supplement" volume contains a summary of the literature reviews made during this study to derive the methods used to analyze the data and to develop the alternative solutions. It also contains a development of the potential economic benefits. The "Field and Laboratory Data" volume contains the results of field and laboratory investigations, including flow rates, quality parameters, and precipitation data, as well as selected hydrographs and quality plots. These latter two volumes have a limited publication and are available for inspection only at the offices of the FWQA and Metcalf & Eddy. Numbers enclosed in parentheses within the text are the references listed in Section 13.

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DESCRIPTION OF THE STUDY AREA

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SECTION 3

DESCRIPTION OF THE STUDY AREA

CHARACTERISTICS OF THE STUDY AREA

The six cities of the East Bay Area that form the study area are located in the northwestern portion of Alameda County on the easterly side of San Francisco Bay, one of the finest natural harbors in the world. West of the study area, the headlands of the San Francisco and Marin peninsulas form the narrow entrance to the San Francisco Bay, known as the Golden Gate, which itself is probably the fundamental geographical factor in the area. Eastward through the Golden Gate come the tide, winds, and fogs of the Pacific, which are decisive factors in the climatology of the area.

As shown in Figure 1, the cities appear to be a single entity, with no visible lines of demarcation between them. Together they cover an urban area approximately 12 miles in length and varying from three to eight miles in width.

Figure 2 shows the boundaries of the study area: the city limits of Albany on the north, Berkeley and Oakland on the east, Oakland on the south, and the San Francisco Bay on the west. The distribution of area (in acres) and population are listed in Table 1.

TABLE 1

AREA AND POPULATION OF THE SIX CITIES IN THE EAST BAY AREA

City	Total area,* acres	Land area, acres	Estimated population 1968**
Alameda	14,790	6,459	70,200
Albany	3,218	1,084	17,200
Berkeley	11,585	6,503	121,500
Emeryville	1,390	760	2,850
Oakland	49,330	35,520	379,500
Piedmont	1,090	1,090	11,500
Total	81,403	51,416	602,750

*Includes portions of San Francisco Bay lying within city limits.

**By the Alameda County Planning Department, as of January 1, 1968.

FIG. I VICINITY MAP OF THE STUDY AREA

FIG 2. BOUNDARIES OF EBMUD, SPECIAL DISTRICT NO.1- STUDY AREA

PRINCIPAL FEATURES

Figure 3 is an aerial view of the study area from a position over the San Francisco Bay. Following along the shoreline from the north to the southeast, some important features are: Berkeley Marina, with its recreation pier and small craft harbor; Aquatic Park on the south Berkeley waterfront just east of Eastshore Freeway (Interstate Route 80); the recently expanded Oakland Marine Terminal Facility; Oakland Inner Harbor, which separates the island city of Alameda from the city of Oakland; the U.S. Naval Air Station on the westerly end of the island; and San Leandro Bay, a shallow body of water connected to the main Bay by a natural channel at the southeast end of Alameda. Nearly all of the shoreline of the East Bay cities runs out westerly in shallow mudflats on a practically level plain, roughly four miles in width, to a depth of 15 feet at mean lower low water.

TOPOGRAPHY, GEOLOGY, AND CLIMATE

The East Bay Area includes an alluvial plain lying between the hills of the Coastal Range along the eastern boundary and the waterfront. This plain rises gently eastward from the shoreline for a distance varying from one to three miles, to the foot of the range of hills forming the eastern boundaries of Berkeley and Oakland. Beyond the plain, the Berkeley Hills rise rather steeply in a series of ridges trending southeasterly. The average crest elevation is about 1,200 feet, and the highest point is Bald Peak at 1,913 feet. In the east Oakland area the highest point is Redwood Peak at 1,619 feet.

Several creeks which traverse the plain from the foothills to the waterfront govern the layout of the drainage and sewerage systems of the various cities.

The Temescal Alluvium is an alluvial plain of simple geologic structure forming part of the East Bay Area. The predominant geological formations are undifferentiated Quaternary (recent) deposits which include sand, gravel, clay, bay mud, recent alluvium and colluvium, and artificial fill. Artificial fills of varying composition are found in strips along almost the entire shoreline, usually constructed over former tidelands.

In the ridge east of Berkeley, the component strata are quite complex, but in general they are sedimentary and metamorphic formations with igneous intrusions and remnants. Behind Berkeley proper, the formations are principally sedimentary sandstones, conglomerates, and shales. Bald Peak is a basalt cap lying on a conglomerate mass of the ridge. Proceeding northeasterly from the alluvial plain in the region of east Oakland, the formation between the approximate elevations of 100 and 200 feet is the San Antonio deposit of alluvial fans. Above this the Leona Rhyolite rises with quite abrupt slopes to the crest. The rate of rise is about 1,100 feet in one and one-half miles

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FIG. 3 AERIAL VIEW OF THE STUDY AREA

Appendix Figure I-1 shows the locations of the various geological formations and artificial fills in the study area, as well as the major and minor fault lines. The Hayward Fault zone contains the only faults known to be active in the area. Appendix Table I-1 lists the geological symbols used in the figure.

The climate has three outstanding features: mild year-round temperatures; abundant rains during the winter; and early morning overcast with almost no rain during the summer. Prevailing westerly winds from the Pacific, where temperatures vary little between winter and summer, keep the winters mild and the summers cool. Sustained periods of either hot or cold weather occur infrequently.

Approximately 90 percent of the average total annual rainfall of almost 18 inches (measured at the Oakland Airport) falls during a six-month period, beginning in early November and extending through the end of April. This period is called the "rainy season" in the report. In the summer, the cooling sea breezes and early morning overcast combine to produce a high relative humidity. Although rain usually falls over the entire East Bay Area during storms, the annual amounts vary from place to place, the hilly portions receiving larger amounts than the lower plain.

The characteristics of the climate for the East Bay Area are summarized in Table 2.

LAND USE

The distribution of major land uses of the cities is listed in Table 3 and shown in detail on maps in Appendix Figure I-2. Most of the industrial sites are located on a strip of land approximately one mile wide extending along the shoreline; in Alameda they are on a narrow strip along the Oakland Inner Harbor and Estuary. Commercial areas, other than the downtown areas of Alameda, Berkeley, and Oakland, consist of strip developments along main arterials. Large portions of the East Bay Area are occupied by the Metropolitan Oakland International Airport, the Alameda Naval Air Station, the Naval Supply Center and Depot, the Oakland Army Terminal, the University of California, the California Schools for the Deaf and Blind, Mills College, and the Oak Knoll Naval Hospital. The zone between the industrial areas and the hills consists of commercial and low-to-high density residential use. The steeper hillsides are usually utilized for low density residential development. Areas of new developments are few. Development of hillside land continues, but bans on Bay filling have curtailed tideland development. The cities are approaching the saturation level as measured by today's standards.

Alameda, Albany, and Piedmont are primarily residential cities, whereas Emeryville is primarily an industrial city. Berkeley has areas of considerable industrial and commercial importance, but its primary characteristic and unique feature is that of an educational center.

TABLE 5

CLIMATE CHARACTERISTICS OF THE EAST BAY AREA

	Precipitation, inches per month			Normal temperature, deg F			Average monthly prevailing winds**		Average sky cover, sunrise to sunset***	Average monthly relative humidity, percent	
	Min.	Max.	Normal	Min.	Max.	Monthly	From	MPH		4 a.m.	4 p.m.
Length of record, years	40	40	40	*	*	*	21	21	25	5	5
January	0.65	8.90	3.83	40.6	55.4	48.0	SE	6.3	6.3	81%	71%
February	0.02	8.53	3.21	42.6	58.7	50.7	W	7.0	5.9	76	67
March	0.04	5.69	2.42	44.7	62.5	53.6	W	8.8	5.8	77	65
April	T	4.60	1.38	47.7	65.5	56.6	W	9.2	5.2	77	62
May	T	3.42	0.65	51.2	68.2	59.7	W	9.9	4.8	80	64
June	0.00	1.21	0.12	54.2	71.3	62.8	W	9.9	4.0	83	67
July	0.00	0.14	T	55.9	72.7	64.3	WNW	9.2	3.6	85	67
August	0.00	0.34	0.03	56.3	72.4	64.2	WNW	9.0	3.9	85	67
September	0.00	3.27	0.20	55.5	74.6	65.1	WNW	7.7	3.5	80	63
October	T	8.56	0.78	51.1	71.1	61.1	WNW	6.5	4.3	79	63
November	0.00	5.64	1.73	45.4	63.9	54.7	WNW	5.9	5.9	79	69
December	0.29	11.29	3.58	41.8	57.1	49.5	E	6.1	6.1	79	69
Yearly averages	.00		17.93	48.9	66.1	57.5	W	7.9	4.9	80%	66%

Note: T - Trace, an amount too small to measure.
* Climatological standard normals (1931-1960).
** The prevailing direction for wind is taken from records through 1963.
*** Sky cover is expressed in a range of 0 for no clouds or obscuring phenomena to 10 for complete sky cover.
Source: U.S. Weather Bureau at Metropolitan Oakland International Airport, Oakland, California.

TABLE 3

DISTRIBUTION OF MAJOR LAND USES FOR THE SIX CITIES IN THE EAST BAY AREA

Major land use	Alameda		Albany		Berkeley		Emeryville		Oakland		Piedmont	
	Acres	Percent	Acres	Percent	Acres	Percent	Acres	Percent	Acres	Percent	Acres	Percent
Incorporated area	14,790	100	3,210	100	11,565	100	1,390	100	49,330	100	1,126	100
Water area	8,331	56	2,138	66	5,072	44	630	45	13,810	28	--	--
Land area	6,459	44	1,068	34	6,503	56	760	55	35,520	72	1,126	100
Circulation (streets, freeways, etc.)	626	10	258	24	1,796	27	180	24	7,700	22	223	20
Commercial	177	3	157	14	267	4	12	2	1,250	4	4	1
Residential	1,558	24	448	41	3,021	46	70	9	11,010	31	746	66
Public use, community facilities, government installations	2,127	37	58	5	819	13	23	3	10,970	30	123	11
Industrial	229	4	31	3	319	5	375	49	1,850	5	--	--
Vacant	1,546	24	135	13	260	4	100	13	2,700	8	30	2
Agricultural	150	2	--	--	--	--	--	--	--	--	--	--
Population	56,200		17,200		121,500		2,450		379,500		11,500	

Within the city is the original campus of the University of California. There is a central commercial district and some strip commercial development along the main traffic arteries. The industrial development is located in the areas between San Pablo Avenue and the waterfront. The development of the Berkeley Marina has added recreational use to the waterfront.

Oakland is, by far, the largest city in the study area and the economic and transportation center of the East Bay. The entire waterfront area is involved in industrial use. There is a large commercial downtown area, as well as strip commercial development along many of the main traffic arteries. Much of the area around the shores of Lake Merritt has been developed for either commercial or high density residential use, with the exception of several public recreational developments. The foothill and sidehill areas are devoted to medium and low density residential units.

GROUNDWATER LEVELS

Figure 4 presents the data that were available for defining the groundwater conditions. Part of the information is a compilation of boring data collected by the U.S. Geological Survey over the past 30 years and cannot be given any great weight because of changing conditions. The most recent and most valid data are those developed during the foundation investigation made for the Bay Area Rapid Transit System.

Discussions were held with various sewer maintenance and construction personnel regarding the possible interference of groundwater during sewer construction. Except in locations near the waterfront, their experience indicated that water levels were below the normal sewer depths and did not represent a problem during sewer construction.

Except for local variations, the groundwater seems to be ten feet or more below the ground, which indicates that groundwater may not represent a substantial factor in infiltration into the sewer system. This conclusion seems to be confirmed by the fact that dry weather flow rates at the water pollution control plant are not abnormally high. Other factors seem to predominate in the initial volume of infiltration, for example, direct connections between storm and sanitary sewers and indirect connections formed by a porous bedding material situated between leaking storm sewers and broken sanitary sewers. Obviously, infiltration from groundwater sources cannot be overlooked because this source probably produces that portion of infiltration that may persist for one or more days after a storm.

SEWER SYSTEM

Many of the street systems in the cities of the East Bay Area have been laid out on the familiar rectangular basis, with streets running directly up the slopes and laterally across them. As a result, most

19

FIG.

SOURCE

LEGEND

12.5 DEPTH TO FREE WATER SURFACE

<u>12.5</u> DEPTH TO FREE WATER SURFACE DURING RAINY SEASON

FIG. 4 GROUNDWATER DEPTHS IN THE STUDY AREA

SOURCE: USGS, SELECTED BORINGS - ALAMEDA COUNTY, 1930-1965 AND BAY AREA RAPID TRANSIT DISTRICT CONSTRUCTION DATA - BORINGS

of the sewer lines run directly down the slopes on considerable grade until reaching flat ground. In the more recently developed residential areas of the middle and upper hills, the streets tend to follow the contours, with relatively few cross-streets up and down the slopes. Consequently, there is no difficulty in providing adequate grades for the sewers in the upper areas.

During early sewer development in the area, sewers were designed to run directly downhill on the shortest possible lines to discharge at the shores of the Bay or its estuaries. As a result, a large number of outlets were discharging untreated domestic and industrial wastes into the receiving waters. The water pollution control plant was built to eliminate this problem. Two interceptors--one from the north and one from the south--collect and carry sewage to the plant. They were placed as close to the waterfront as possible to connect the existing outfalls without necessitating reconstruction of the existing system. The locations of the interceptors, the pumping stations, and the water pollution control plant are shown in Figure 5.

Appendix Figure I-3 shows the size, number, and shape of the various drainage areas tributary to the interceptor sewers. As indicated by their shape, the drainage areas conform to the natural geographical conditions.

The interceptor system consists of approximately 21 miles of reinforced concrete pipe sewers, varying in size from 12 to 108 inches in diameter, and 12 pumping stations, varying in capacity from 0.5 to 57 mgd. Two pumping stations at the water pollution control plant lift the incoming raw sewage from the interceptors into the treatment units and pump the treated waste from the plant when the flow rate exceeds the gravity-flow capacity of the outfall line.

The 12 pumping stations are automatically controlled throughout the year, except during peak storm flows when Pump Station H is manually operated or shut down to prevent overloading of the water pollution control plant. The interceptor system is pumped down to minimum levels each week to create velocities high enough to remove solids deposited within the system during periods of low flow.

The physical condition of the sewer system can be described by reference to age, pipe materials, and integrity of pipe walls and pipe joints. Some of the older sewers in the systems date back almost 100 years. Many of the early sewers were constructed by land developers, with little or no control by the cities or agencies responsible for this service; therefore, the quality of construction left much to be desired. The major system of combined sewers was constructed prior to 1938. As has been mentioned, the sewer separation and reconstruction began after issuance of the "1941 Sewage Report" (1). The majority of this work was completed by the early 1960's, although some work has continued each year. Additional effort is necessary before all of the combined sewers are separated.

FIG. 5 SEWAGE DISPOSAL SYSTEM FOR EBMUD, SPECIAL DISTRICT NO. 1

Vitrified clay pipe has been used almost exclusively for the construction of sewers, although some of the large sewers were constructed of brick or concrete. The joints used in the earlier clay pipe sewers were made of cement mortar. In addition to being rigid, they were unsatisfactory because all too often insufficient care was taken to close the invert portion of the joint. Later construction made use of sulphur compound joints, which were also rigid, and pipe settlement caused failures. Various other jointing compounds were used until the superior plastic joint became available around 1960. This plastic joint is now used predominantly.

Figure 6 shows four photographs of sewer failures which were provided by the City of Berkeley. The pictures are examples from a photographic survey of the sewers located in Keith Street (between Euclid and Brett Harte Road) and Keeler Street (between Marin and Forest Hill) in Berkeley. They illustrate the worst conditions that were found along those sewers, but these were not all of the failed conditions. Copies of the survey reports are included in Appendix I. As a result of these reports, these particular sewer reaches have been replaced. The question now is how many other reaches of sewer throughout the East Bay Area are in similar condition?

Sewer construction and maintenance practices found during an interview survey are summarized in Table 4. Some of the more interesting items are noted below.

All cities now require that the maximum distance between manholes not exceed 300 to 400 feet. Prior to 1950, Alameda allowed 450 feet, Berkeley and Piedmont allowed 600 feet, and Emeryville allowed 1,000 feet.

All cities except Oakland indicated that sewer separation is between 95 and 100 percent complete. Oakland indicated that its sewer separation is 90 to 95 percent complete at present, but that progress is being made each year.

Oakland was the only city to report more than ten catchbasins attached to sanitary sewers; in fact, Oakland estimated there were more than 100.

Albany and Piedmont were the only cities to report no relief overflows existing between sanitary sewers and storm drains. Alameda and Emeryville reported fewer than five, while Berkeley indicated in excess of 50, and Oakland, 20 or more.

Berkeley, Oakland, and Piedmont conduct sewer replacement programs on a year-round basis. Alameda, Albany, and Emeryville only replace failed pipe.

All cities except Emeryville indicated that flooding occurs during rainstorms; however, in all cases, it was considered a mild problem.

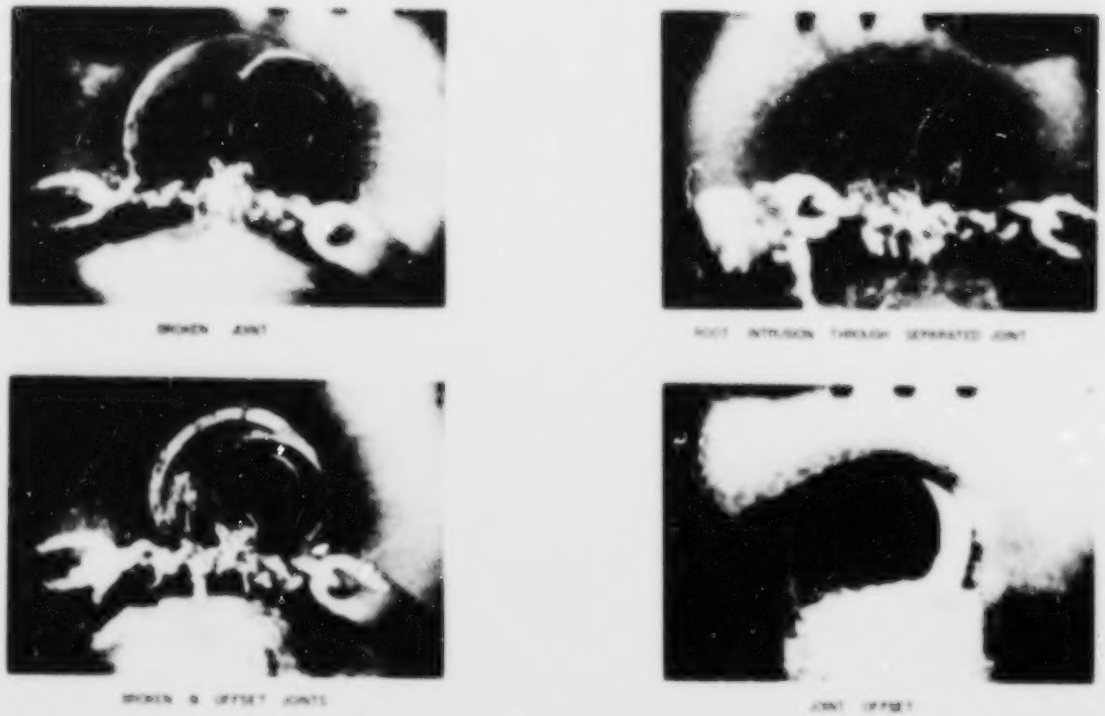

FIG 6 EXAMPLES OF FAULTY PIPE WALLS AND JOINTS

SOURCE: CITY OF BERKELEY SEWER MAINTENANCE DEPARTMENT

TABLE 4

RESULTS OF SEWER CONSTRUCTION AND MAINTENANCE SURVEY, EAST BAY AREA

Question*	Albany	Berkeley	Emeryville	Oakland	Alameda	Piedmont
1. Specify vitrified clay pipe as standard for sewers with plastic joints?	X	X	X	X	X	X
2. Allow asbestos cement pipe?					X	
3. Allow cast iron or corrugated pipe for special requirements?		X		X		X
4. Specify special bedding under pipes?	X	X	X		X	X
5. Allow curvilinear sewers:						
a. horizontal curves?				X		X
b. vertical curves?				X		X
6. Maximum manhole spacing = 300-400 ft?	X	X	X	X	X	X
7. Disallow roof connections to sanitary sewers?	X	X	X	X	X	X
8. Test for leaks on new sewer construction?	X		X	X	X	X
9. Specify air testing for sewers?				X		X
10. Infiltration allowance = 500 gallons per inch diameter per mile per day?	X		X		X	
11. Have established maintenance program?	X	X	X	X	X	
12. Does flooding occur occasionally during rainstorms?	X	X		X	X	X

Note: X indicates a yes answer to the question, and a blank indicates either no or some other condition.
*Questions were asked during interviews with public works representatives of each city.

All cities indicated they had few problem areas. Berkeley and Oakland indicated that tides have an effect on the sanitary sewers, but only if high tides occur during periods of heavy rainfall.

WATER POLLUTION CONTROL FACILITIES

The water pollution control plant operated by EBMUD, Special District No. 1 was designed to provide primary treatment for an average dry weather sewage flow of 128 mgd and a maximum flow of 291 mgd. Facilities are provided for prechlorination, screening, grit removal, sedimentation, and sludge digestion. In 1968 postchlorination facilities were added. The treated waste water and the digested sludge solids and supernatant are disposed through an outfall sewer which discharges into the offshore waters of San Francisco Bay.

Main Pumping Station

Upon arrival at the plant, the sewage flow first enters the main pumping station where it is immediately chlorinated to control odors. Next, five mechanically-cleaned bar screens with one-inch openings remove sticks, rags, and other trash from the sewage flow, which is then lifted by five 42,000-gpm pumping units into the grit chambers.

Grit Chambers

Grit, composed of sand, gravel, and other heavy matter, is settled out as sewage flows at reduced velocity through the grit chambers. These chambers consist of five channels, each 60 feet long and 12-1/2 feet wide, with an average water depth of nine feet. The settled grit is continuously collected and mechanically removed, and then hauled to a dump site where it is buried.

Sedimentation Tanks

From the grit chambers, the sewage flows by gravity to ten sedimentation tanks, each 180 feet long and 35 feet wide, with an average water depth of 11 feet. It is retained in the tanks for approximately one hour during which the settleable organic solids are removed. The heavier solids settle to the bottom of these tanks and accumulate in a form of sludge, which is mechanically collected and pumped to the sludge digestion tanks for treatment. The floating solids and the oil and grease that rise to the surface of the tank are skimmed off and incinerated.

Sludge Digestion Tanks

The sludge from the sedimentation tanks is transferred to the sludge digestion tanks and held at a temperature of 95 deg F for approximately 30 days. During this period, it is converted by biochemical action into an innocuous humus-like solid residue, liquid, and gas. The

digested sludge is discharged into the outfall sewer. The supernatant (the liquid residue) is returned to the headworks. The gas is collected, and some of it is used as fuel to heat the sludge digestion tanks and plant buildings; the remainder is burned in waste gas burners.

Effluent Pumping Station

The effluent or liquid overflow from the sedimentation tanks flows by gravity to the effluent pumping station. From here, it either flows by gravity or is pumped by three 70,000-gpm pumping units into the outfall sewer.

Outfall Sewer

The outfall sewer is three miles long and has a diameter of 108 inches. It conveys the effluent and the digested sludge to the point of final disposal one mile offshore and 45 feet below the surface of San Francisco Bay. The end of the diffuser is located approximately 540 feet south of the Bay Bridge. Here the effluent is rapidly dispersed and diluted by tidal action.

CHARACTERISTICS OF THE STUDY SUBAREAS AND CONTROL AREAS

Ten subareas in Berkeley and Oakland were selected for intensive study:

Berkeley subareas

1. Benvenue Avenue
2. Carleton Avenue
3. Glen Street
4. Spruce Street

Oakland subareas

1. Capistrano Drive
2. High Street
3. Nineteenth Street
4. Pump Station J
5. Skyline Boulevard
6. Trestle Glen Road

These subareas, shown on Figure 7, were selected because together they typify the various major land use categories, they represent both old and new sewers, and they include both flat and hillside terrain. Other considerations were the ease with which samples of flow data could be collected and the characteristics of the area that would permit generalization of subsequent data to the total study area or to areas outside the study area. Table 5 lists land use, topography, and sewer information for the subareas. Additional information on each subarea follows.

BERKELEY SUBAREAS

Benvenue Avenue

The Benvenue Avenue sanitary sewers consist of predominantly older clay pipe with cement mortar joints. Direct connections are suspected of contributing to the infiltration problem during rainstorms, but a roof

TABLE 8

LAND USE, TOPOGRAPHY, AND SEWER DATA FOR EAST BAY SUBAREAS

	Land Use							Sewer Data		
Subarea	Category*	Area, acres	Number of houses	Number of buildings excluding houses	Roof area, acres	Street and parking, acres	Topography	Age**	Estimated length, feet	Size, inches
Berkeley										
Benvenue Avenue	MD RES	106	295	35	25	11	Flat	Old	13,700	4-12
Carlston Avenue	INDUS	28	--	45	14	6	Flat	Old	4,270	6-10
Glen Street	LD RES	61	1,420 (Glen Street and Spruce Street combined)	--	102	65	Hilly	Old	26,620	4-15
Spruce Street	LD RES	459					Hilly	Old	104,900	4-15
Oakland										
Capistrano Drive	LD RES	95	570	1	20	23	Flat	Old	18,470	6-15
High Street	INDUS	74	--	22	8	12	Flat	New	4,560	8-12
Sixteenth Street	COMM, HD RES	61	--	90	28	16	Flat	Old	15,470	6-14
Pump Station 2	INDUS	[illegible]	--	24	17	7	Flat	Old	--	8-18
Skyline Boulevard	LD RES	82	135	--	9	12	Hilly	New	13,250	6-8
Trestle Glen Road	LD RES	645	2,600	--	89	75	Flat to hilly	Old	148,340	6-14

*HD RES = High density residential; MD RES = Medium density residential; LD RES = Low density residential; COMM = Commercial; INDUS = Industrial.

**The term old means any area constructed prior to 19[illegible] when the cities adopted the plastic joint clay pipe as a standard for sewer construction.

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BENVENUE AVE SUBAREA
GLEN ST SUBAREA
SPRUCE ST SUBAREA
TRESTLE GLEN RD SUBAREA
CARLETON AVE SUBAREA
NINETEENTH ST SUBAREA
PUMP STATION SUBAREA

30

FIG 7 LOCATION MAP OF THE SELECTED STUDY SUBAREAS

leader survey was not undertaken as part of this investigation; therefore, this suspicion was not specifically confirmed.

This subarea is located south of the University of California campus where the soils are usually clayey and shrink or swell with seasonal moisture changes. Drainage is only fair since there is a slowly permeable, clayey subsoil. Slope stability and foundations vary from poor to fair.

Land use is primarily single-family residential; however, portions are zoned for restricted two-family residential, restricted multiple-family residential, and multiple-family residential uses (primarily rooming houses for students at the University of California). The area was therefore assumed to fit the category of medium density residential land use.

Carleton Avenue

The Carleton Avenue sanitary sewers are quite old and have poor joints. This subarea was selected because it is an isolated industrial area consisting of manufacturing and pharmaceutical production facilities. Consequently, most of the sewage is industrial rather than sanitary in nature. Daily flow in the sewer is subject to the production cycles of the manufacturers and producers in the area. It is believed that few illicit connections exist, hence the infiltration problem may not be as acute as in the residential sections of the city.

This subarea is located on the flat plain east of Aquatic Park in Berkeley. The soil is a fine-textured alluvium which is deep and rather poorly drained, with an intermittent water table below 48 inches. There is no erosion hazard with this type of soil, which includes gravel, sand, and clay as well as Bay mud and artificial fill. The soil mantle may be as much as three feet thick. The clay shrinks and swells with seasonal moisture changes and may cause damage to existing buildings and sewers.

Glen and Spruce Street

The Glen and Spruce Street subareas are identical in land use and topography. Initially, only one subarea was contemplated, but two stations were necessary to measure and sample the flow. Throughout the remainder of this report these areas will be discussed separately.

The sewerage system consists of old clay pipe sewers with poor joints. Numerous direct connections are suspected of contributing to the infiltration problem during rainstorms. During the middle 1950's, a sewer system study was conducted by the City of Berkeley to determine the location of roof drain and area drainage connections. When the study was completed, homeowners were notified to disconnect the roof connections; however, no follow-up study was undertaken, and no check was made to determine whether all of these connections had been discontinued.

These subareas are located in the northeastern portion of Berkeley near the crest of the ridge which forms the boundary of EBMUD, Special District No. 1. This portion is on both the mild and steep slopes of the Berkeley Hills. Some of the streets are laid out on the familiar rectangular grid system, but, in the steeper areas, they tend to follow the contour line. The soil is primarily a loam cover over a sandstone base, and the depth of the parent rock or sandstone varies from a few inches to three feet. Drainage varies from good to very good. Since these subareas are in the immediate vicinity of the Hayward Fault, ground movement causes sewers to be broken, hence an abnormal condition of infiltration and root intrusion exists. The sewers are generally quite shallow. Several springs exist because of the very good drainage feature of the soil, which could also contribute substantially to the infiltration problem. During this investigation, a portion of the sewer system within the subareas was replaced. In that portion many of the sections of clay pipe had been either crushed or sheared as a result of land movement or poor installation. The groundwater is near the surface since the soil layer is fairly thin. The erosion hazard varies from high to very high because the soils are not cohesive in nature.

Land use is primarily confined to single-family residential although there is some two-family residential use. Because the upper reaches of these subareas are quite steep, the population density decreases toward the ridge.

OAKLAND SUBAREAS

Capistrano Drive

The Capistrano Drive sewerage system consists of old clay pipe sewers with poor joints. Numerous direct connections are suspected of contributing to storm water infiltration. The sanitary sewers differ from those in other subareas in that they generally are not located under pavement but along back lot lines or outside the travelled way. Therefore, more joints of these sewers are subject to possible groundwater infiltration than would normally be the case.

This subarea is located in east Oakland on the south quarter of EBMUD, Special District No. 1 near San Leandro Creek. It is primarily limited to single-family residential use. The land is a flat plain having a loamy surface with poorly draining clay and silt subsoil. There is no erosion problem.

High Street

The High Street sanitary sewer system consists of new clay pipe with plastic joints. Few direct connections are suspected since the sewer has been rebuilt within the last four years (1966). This sewer serves a new industrial development in addition to the older industries.

The subarea lies along the Oakland Inner Harbor in south central Oakland. The soil is composed entirely of artificial fill placed over tidal flats or shallow waters. In general, this type of soil has poor drainage. The sewers were laid above the groundwater table, hence should not be subject to continuous infiltration.

Nineteenth Street

The Nineteenth Street sewerage system consists of old clay pipe sewers with poor joints. Many of the manholes are made of brick. Direct connections are suspected to contribute to the storm water infiltration. Past studies conducted by the City of Oakland have indicated that illegal (forbidden by local Plumbing Code) roof and floor drain connections exist.

This subarea is in the flat plains southwest of Lake Merritt in central Oakland. The soil is a clayey mixture with slowly permeable clay subsoil in poorly draining loamy sand. Since the land is flat and the soil exhibits cohesive properties, there is little erosion danger. Because the location is just inland from the Oakland Inner Harbor, the groundwater table is influenced by the tides to some degree and by the water level in Lake Merritt.

This subarea is a concentrated commercial and high density residential area which is not limited by the characteristics of strip development, found elsewhere in the study area. It is at the low point of the drainage shed adjacent to Lake Merritt, a controlled level lake with beautifully landscaped shores near downtown Oakland, and includes a portion of the business district, along with several exceptionally fine hotels and apartments. The land along the lakefront is occupied by high-rise apartments. This subarea is of particular concern to the City of Oakland, because overflows from sanitary sewer manholes in the vicinity of Lake Merritt would flow directly into the lake.

Pump Station J

The Pump Station J subarea is an older, medium-to-light industrial area, and the sanitary sewers consist of old clay pipe generally below the groundwater table.

Located in west central Oakland along the Oakland Inner Harbor, this subarea is generally a flat plain having a loamy-sand surface and subsoil which has poor drainage. The water table is subject to tidal variation. There is no erosion problem except for tidal action.

Industrial areas that could be isolated for purposes of flow measuring and sampling were difficult to find. For this reason, the Pump Station J subarea actually consists of two contribution areas. Flow measured at the Pump Station is a mixture from both areas. Therefore, a second station was installed to measure and sample the flow from the other area. This station is on Kennedy Street and will be referred to by that

name. The area contributing to Kennedy consists of a conglomeration of land uses, so that the volume of flow and mass of pollutants contributed by the area served by Kennedy must be subtracted from the corresponding values measured at Pump Station J.

Skyline Boulevard

The Skyline Boulevard sewers are clay pipe with plastic joints installed since 1965. There are very few direct connections which permit storm water to enter the sanitary sewer system.

This subarea lies along the crest of the east Oakland Hills at the south border of EBMUD, Special District No. 1 and is characterized by steep slopes and sparce vegetation. There is a loam topsoil with a sandstone base throughout the subarea; the topsoil varies from a few inches to three feet in thickness. Drainage is good to very good, and the erosion hazard varies from high to very high. Land use is single-family residential; development has taken place since 1965.

Trestle Glen Road

The Trestle Glen Road sewerage system consists of old clay pipe with poor joints. Numerous direct connections are surpected, since the manholes of the lower section of the sewer system are subject to overflowing. Rainfall intensities as low as 0.05 inch per hour may cause the sewers to overflow. The basements of several homes in the lower portion of this subarea are subject to flooding, since the basement drains are connected directly to the sanitary sewers system without backwater valves.

This subarea is located partly within Piedmont and partly within central Oakland, east of Lake Merritt. The Oakland portion is relatively flat, while the Piedmont portion tends toward steep hills. The surface soil is silt and loam; the subsoil is a silty-clay loam. Depth to bedrock is approximately 20 inches. The soil is well drained, and there is a moderate erosion hazard. This subarea consists of a portion of the Trestle Glen or Indian Creek drainage area which contributes to the northeastern arm of Lake Merritt. Overflows from sanitary sewers in this area have traditionally caused pollution problems in Lake Merritt.

Land use is predominantly single-family residential. Since the upper reaches of this subarea are quite steep, the population density decreases and the lot size increases.

CONTROL AREAS

In addition to the ten study subareas, three large drainage areas were selected as "control areas" for collecting data needed to substantiate

the procedures used for extrapolating subarea data to the overall study area. Three major flow measuring and sampling stations were chosen:

1. Pump Station A, located in the northwest corner of Albany.

2. Pump Station H, located near the High Street Bridge in south central Oakland.

3. The main pumping station at the EBMUD Water Pollution Control Plant.

These stations serve progressively larger and more complex areas. Pump Station A serves only one drainage area consisting of Albany and parts of north Berkeley. Pump Station H serves six drainage areas, including the Metropolitan Oakland International Airport. The main pumping station serves all of the drainage areas in the study area.

SECTION 4

FLOW MEASURING AND SAMPLING PROGRAM

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SECTION 4

FLOW MEASURING AND SAMPLING PROGRAM

The equipment and procedures used during the flow measuring and sampling phase of this investigation are described in this section, together with some of the problems encountered in their use. The resulting data are discussed in Section 5.

As discussed in Section 2, the approach devised for determining the amount and effects of storm water infiltration involved the development of detailed infiltration relationships for the subareas, followed by an extrapolation or application of these relationships to the whole study area. Inherent in the selection of the subareas was a consideration of an available or convenient flow measuring and sampling location. In several cases other considerations prevailed, and as a result, several locations were in manholes situated near the centers of streets, thereby complicating the procedures.

Flow measuring and sampling involved both wet and dry weather conditions. Measurements during wet weather included both the rainfall and the discharge rate from each subarea so that a relationship between the two could be determined. Sampling during rainstorms was done to determine the variation in quality caused by the infiltration of storm water. Dry weather flow measuring and sampling was necessary to characterize the normal or base conditions needed for defining the extraneous flow quantities produced by storm water infiltration and the effect of extraneous flows on the normal water quality parameters.

Recording flow measuring equipment was installed in manholes at the point of discharge from each subarea. Recording ammeters were installed at pumping stations where flow measuring devices did not exist.

Sampling during wet weather conditions was done on a manual basis, except at the water pollution control plant. Every effort was made to begin sampling at the beginning of a substantial storm. This, of course, was difficult because there was no way to predetermine the intensity and duration of the rainfall. It was also difficult to begin the sampling immediately after the beginning of a storm because of both the distance to be travelled by the sampler and the irregularity of the storms. Consequently, many samples were taken of relatively small storms or of only the latter portions of storms. The sampling period was predetermined to extend five hours after the beginning of a storm. In some cases, this was extended to cover the duration of the extraneous flow.

Sampling during dry weather conditions, accomplished by automatic equipment at regular intervals throughout the day, was done so that

samples taken during storm conditions could be compared and the effects of storm water infiltration on the quality could be evaluated.

Rain gages, both recording and tube types, were installed in the subareas to provide the detailed data needed to characterize the intensity and duration of the storms. These data were then used to:

1. Aid in defining the source(s) of infiltration by indicating the time required to produce infiltration in the sanitary sewers after significant rainfall began and by indicating the time required for infiltration to cease once significant rainfall stopped.

2. Determine the relationships between rainfall and infiltration in the subareas and, on the basis of this information, estimate the infiltration outside the subareas.

3. Examine the areal and temporal variations for the storms occurring during the study period as an aid in selecting design storms.

Charts from recording rain gages provided the times at which the storm began and ended as well as the intensity variations in between. These times were then used to estimate the time of concentration, computed as the time between the beginning of the storm and the arrival of the peak rate of flow at the point of discharge from the subarea.

FLOW MEASURING

A preliminary review was made of the sewerage system to select flow measuring locations that were not likely to have either surcharging or an intersection of several pipes. This latter requirement was easily determined by reviewing the as-built drawings and by investigating the actual location, but the question of surcharging was not so easily resolved. A surcharged condition would result in no flow measurement from almost any kind of standard sewer flow measuring equipment available.

A search for a new means of measuring flow in the sewer, both under normal and surcharged conditions, was fruitless. Therefore, standard types of flumes and weirs were selected, and in locations where surcharging was suspected, a weir was used with level measurement on both sides, so that in case of submergence at least an approximation of the flow could be made.

Table 6 lists the 16 locations at which flow measurements and samples were taken, and the types of primary and secondary devices that were used. The conditions at these stations are described in the following discussion.

TABLE 5

LOCATIONS AND TYPES OF EQUIPMENT
USED IN FLOW MEASURING AND SAMPLING PROGRAM

Location	Primary devices	Secondary devices
Metering at New Stations		
Berkeley		
1. Benvenue Avenue	Palmer-Bowlus flume	Taylor*
2. Carleton Avenue	90 deg V-notch weir	Stevens**
3. Glen Street	90 deg V-notch weir	Taylor
4. Spruce Street	90 deg V-notch weir	Taylor
Oakland		
5. Capistrano Drive	90 deg V-notch weir	Stevens
6. Nineteenth Street	90 deg V-notch weir	Taylor
7. Kennedy Street	90 deg V-notch weir	Stevens
8. High Street	Palmer-Bowlus flume	Taylor
9. Skyline Drive	90 deg V-notch weir	Taylor
10. Trestle Glen	Palmer-Bowlus flume	Taylor
Metering at Existing Stations		
11. Parkridge Lift Station	--	Amprobe***
12. Pump Station A	--	Amprobe
13. Pump Station J	--	Amprobe
14. Pump Station H	Venturi meter	BIF
15. Water Pollution Control Plant	Pump running time recorder	--
16. Overflow Structure	--	Taylor

Note: Data from High Street and Parkridge Lift Stations were not usable. Mention of commercial products does not imply endorsement by the Federal Water Quality Administration.

*Taylor Pressure Recorder, Series 79J, with 1 or 2 pens, manufactured by Taylor Instrument Company.

**Stevens Model 2A35 liquid level recorder, manufactured by Leupold & Stevens Instruments, Inc.

***Amprobe Model LAA81 or -82 recording ammeter, manufactured by Amprobe Instrument Division of Soss Manufacturing Company.

METERING AT NEW STATIONS

Palmer-Bowlus flumes were installed in three (1, 8, and 10 in Table 6) of the ten metering stations established specifically for this study. These flumes were constructed of stainless sheet steel and designed to fit the respective sizes and shapes of the sewers. They were carefully fitted into the sewers and leaks were stopped by caulking. The flumes were mounted in the outlet sewer from the manholes so that head measurements could be made at the proper distances upstream from the throats.

Figure 8 shows the typical installation of a Palmer-Bowlus flume in a manhole with the channel extensions which effectively increased the throat depth of the flume. Without these extensions the water would have spread out in the manhole after reaching the spring line of the sewer, rendering the flume useless.

At the remaining seven new metering stations, 90 degree V-notch weirs were installed. These weirs, specially designed for ease of installation, were constructed of marine plywood and covered for additional water resistance with two coats of polyurethane finish. Figure 9 shows the design of the V-notch weir in the assembled condition. A channel closure was provided so that it could be easily slipped down into the flowing sewage and quickly bolted in position, after installing and sealing the actual weir plate.

Stevens type 2A35 liquid level recorders were used at three of the weir locations (see Table 6). These recorders were selected for their ability to record two liquid levels simultaneously because submergence of the weir was anticipated. The Stevens recorders were located at meter installations that were not subject to traffic, since they had to be mounted directly over the manhole. They were housed in a wooden enclosure consisting of a rectangular box with two shelves. The entire enclosure was anchored to prevent loss or damage to the equipment.

Taylor recorders were used at all of the other metering stations. The liquid level sensing consisted of measuring the back pressure from a continuously purging bubble system. Nitrogen was used for the gas. The recorders and auxiliary equipment were housed in a wooden instrument enclosure and mounted behind the curb for protection. Figure 10 shows the installation details for a weir in a manhole with the metering instrument mounted behind the curb. The flexible plastic tube for the bubbler was threaded through a buried one-inch diameter conduit from the instrument enclosure, under the street pavement and out to the manhole. A one and one-half inch diameter suction pipe was installed in the same trench with the bubble tube conduit to allow sampling at the curb, which was accomplished with a positive displacement, manually operated pump. These suction pipes were provided only at meter locations where heavy traffic was expected during periods of sampling. In other locations, the samples were taken manually by insertion of a container down through the manhole opening. A third conduit was also

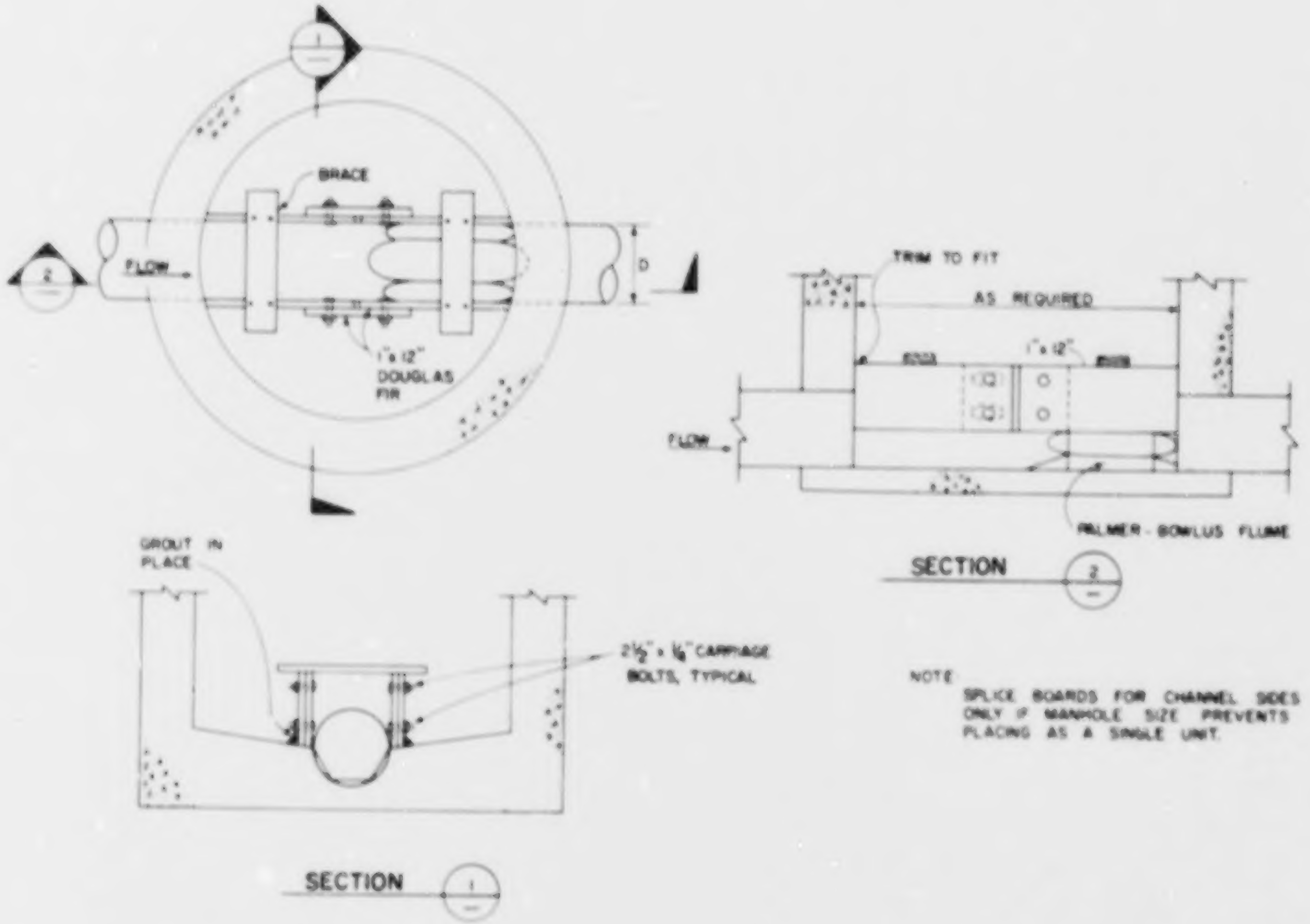


FIG. 8 TYPICAL PALMER - BOWLUS FLUME INSTALLATION

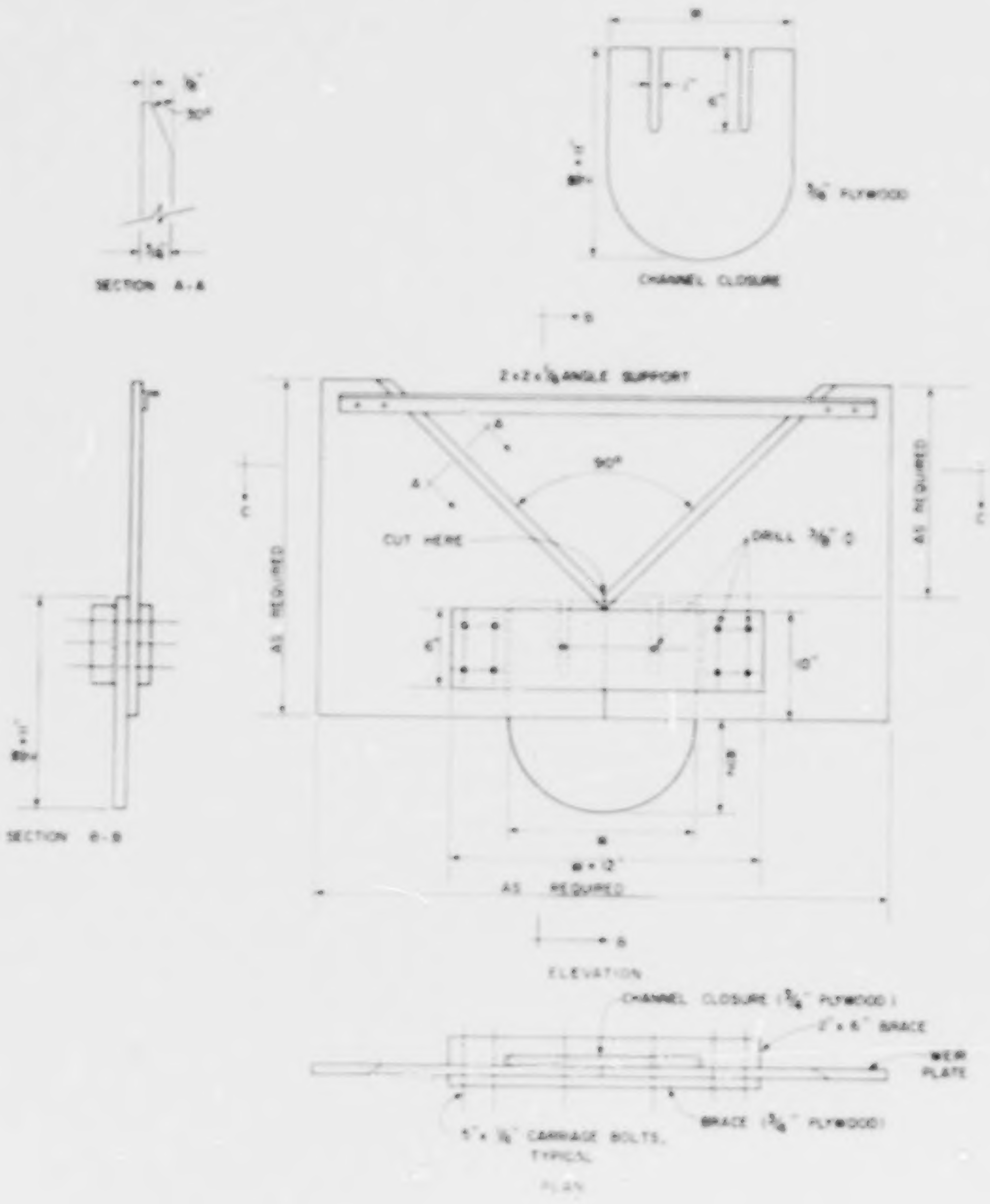
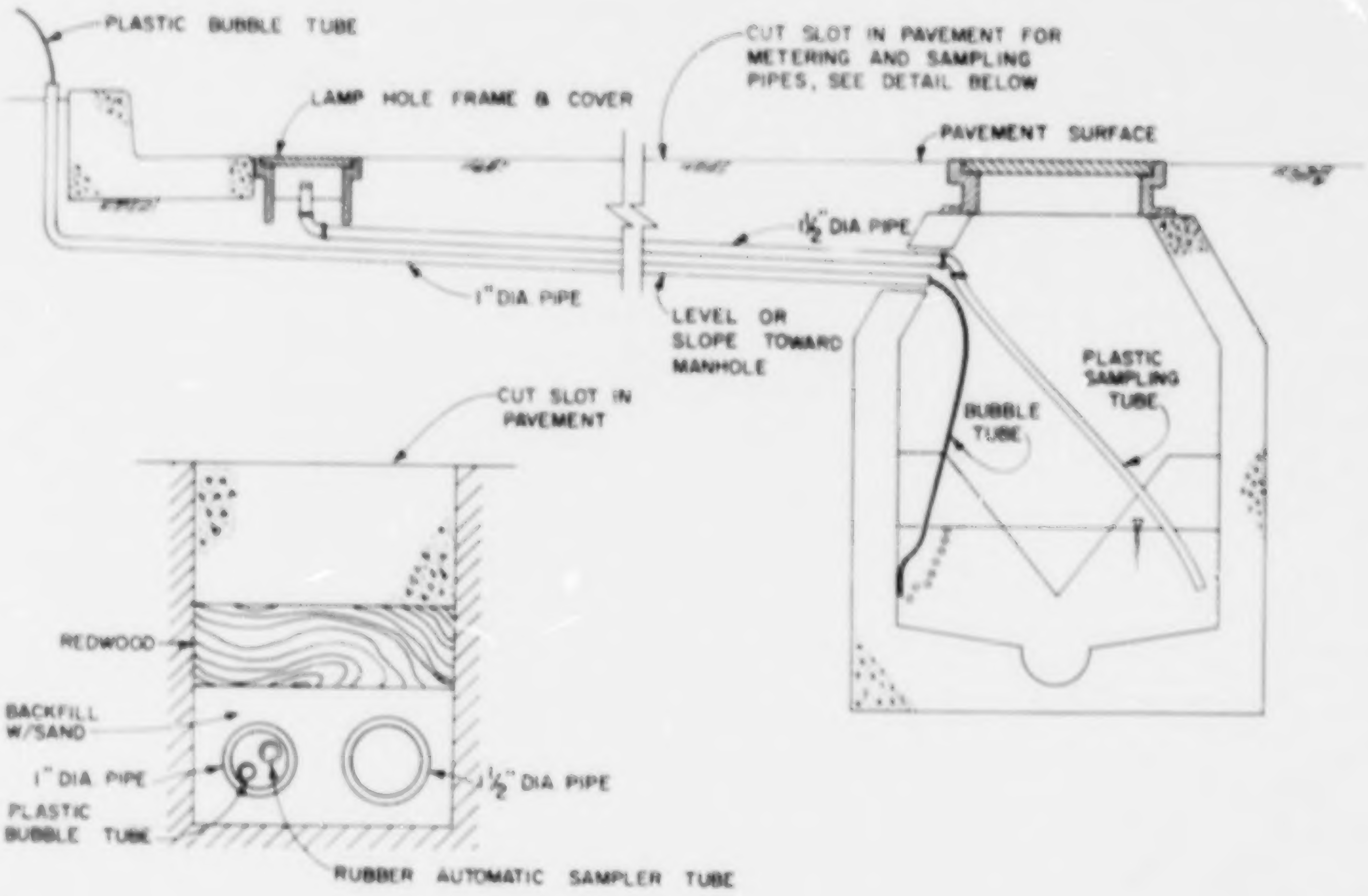


FIG 9 V-NOTCH WEIR PLATE DETAILS



PIPE SLOT DETAIL

FIG. 10 TYPICAL V-NOTCH WEIR INSTALLATION

installed through which the automatic sampler suction tubing was later threaded. Figure 11 shows photographs of typical metering installations.

Liquid levels were extracted from the recording charts at 30-minute intervals, and the actual flow rates were computed by applying the head-discharge relationships for the respective primary device. A typical base flow curve representing the variation of flow rates during the day was developed by averaging the values for the same time of day over ten week days and again over ten weekend days. The extraction of wet weather flows was similar to that for dry weather flows except that averaging over several days, of course, was not possible. The extraneous flows for each individual storm were then computed by subtracting the appropriate period of base flow data from the wet weather flow rates.

Flow measuring equipment was installed at the High Street subarea. Unfortunately, the intermittent discharges from a nearby pumping station resulted in peak flows that prevented proper interpretation of the data. Therefore, the flow data from this station was eliminated and no samples were taken.

METERING AT EXISTING STATIONS

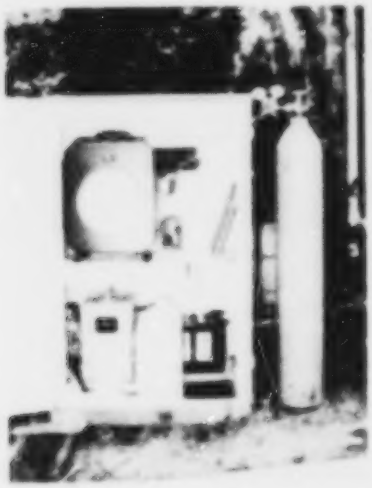
Parkridge Lift Station

The Parkridge Lift Station was considered necessary because it contributed sewage from a newly developing subdivision to the Skyline Drive subarea. It was expected to contribute substantially to the base flow of the subarea. As the study progressed, however, it was found that the flow contribution was negligible. Therefore, the Parkridge Lift Station will not receive further discussion.

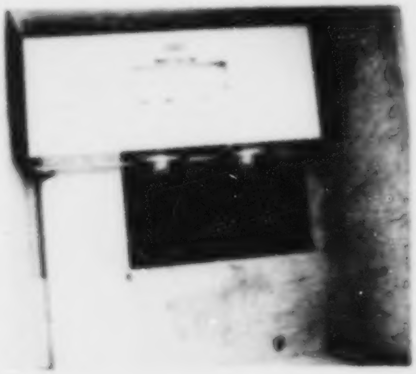
Pump Station A

Pump Station A has four single-speed pumps with equal discharge characteristics. The station is a standard wet well - dry well design with the pumps mounted in the dry well below the ground level and the motors mounted directly above, on the ground level floor. In case the pumps are unable to handle the flow, either because of quantity or mechanical malfunction, an overflow is provided to discharge the flow to Cerrito Creek.

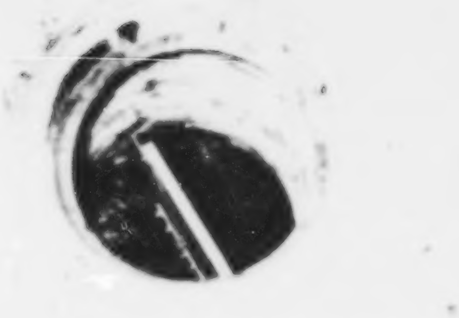
The station is equipped with a wet-well liquid level recorder having a seven-day rotation, circular chart from which flow determinations were virtually impossible because the only information provided was an approximation of the increment of time between pump starts and stops. A determination of which pump or how many pumps were running at any time was difficult at almost all flow rates. Therefore, a recording ammeter was attached to the pump electrical leads to record the instantaneous current drawn by the pumps. The total number of pumps running



INSTRUMENT ENCLOSURE SHOWING TAYLOR RECORDER AND DRY WEATHER SAMPLING EQUIPMENT WITH NITROGEN CYLINDER REMOVED



INTERIOR OF INSTRUMENT ENCLOSURE FOR STEVENS RECORDER



90° V-NOTCH WEIR PLATE INSTALLED IN MANHOLE

INSTRUMENT ENCLOSURE FOR STEVENS RECORDER OVER MANHOLE

FIG. 11 TYPICAL METERING INSTALLATIONS

at any given time and their respective running time was then easily determined. The discharge rate for each pump or combination of pumps was selected from a system-head curve for the station. This curve was developed from information on the physical plant layout and discharge piping, together with the measured head-to-discharge relationship provided by EBMUD, Special District No. 1.

From the ammeter recordings, the operating time for each pump was determined during each one-hour period throughout the day. Applying the appropriate pump discharge rate for the running times resulted in average station discharge volumes for each hour during the day, and the half-hour values were computed by interpolation. The typical base flow values were obtained by averaging the rates for the same time of day over ten week days and again over ten weekend days. These base flow values were used in computing the extraneous flows. The extraction of wet weather flows was similar to that for dry weather flows.

The relief overflow at the pumping station handles flows in excess of the pump capacity during periods of high flow. This overflow consists essentially of a broad-crested weir formed by boards. A rating curve for this weir and the wet-well liquid level recorder permitted computation of overflow quantities. These quantities were then added to the appropriate wet weather flows. The excess flow drains through a flap gate into Cerrito Creek, adjacent to the pump station, and directly into the near-shore water of San Francisco Bay.

Pump Station J

This station is similar in design and construction to Pump Station A, except that it is substantially smaller. The flows here were determined in the same way as at Pump Station A; however, the computation was simplified because there was no overflow or liquid level recorder.

Pump Station H

Pump Station H has a peak pumping capacity of 57 mgd. It is the largest pump station along the interceptor system, with the exception of the main pumping station at the water pollution control plant.

The three pumps at this station are driven by two-speed motors. The normal sequence of operation for the pumps is as follows: one pump at low speed, one pump at high speed, two pumps at low speed, two pumps at high speed, three pumps at high speed. Pump Station H is run automatically, as are all the other outlying pump stations, except during periods of storm flow. It is the only pump station that is operated manually during wet weather conditions, except the main pumping station. Pump Station H may be shut down entirely when the wet weather flow rates approach the maximum pumping capacity at the plant. When this occurs, the flow at Pump Station H is backed into the interceptor and tributary trunk sewers and ultimately overflows at an overflow

structure located on Elmhurst Creek near the Oakland - Alameda County Coliseum complex.

Pump Station H is equipped with both a wet-well liquid level recorder and a flow recorder. The primary device for the flow recorder is a Venturi flow tube mounted in the force main downstream from the pumps. Both the flow rate and the liquid level are recorded on a 24-hour rotation, circular chart that is changed daily at 8 a.m.

Because the flow meter is mounted on the discharge manifold of the pumps, it will measure only the pumping rate at any given time and hence does not directly reflect the variation of influent flow rate throughout the day. Therefore, instantaneous hourly values were read from the charts, and half-hour values were derived by interpolation. These half-hour values of flow rate were further smoothed by averaging the values for the same time of day over ten week days and again over ten weekend days. The resulting values were then assumed to represent the base flow which was subtracted from the wet weather flows to provide the extraneous flow rates during a storm. Wet weather flows were derived in a similar manner, in that rates were extracted from the charts at one-hour intervals and values for the half-hour intervals were interpolated. This tended to smooth out the erratic flow rate which reflected the cycling of the pumps.

Water Pollution Control Plant

The influent pumping station at the water pollution control plant lifts the incoming sewage from the interceptors up into the grit chambers. The pumps are preceded by one-inch clear opening, mechanically-raked bar screens. There are five raw sewage pumps: three are driven by single-speed motors, two by two-speed motors. Each pump discharges directly into individual grit chambers. With all five pumps operating at full speed, the pumping capacity is slightly less than 291 mgd.

The station is equipped with individual flow meters on each pump discharge which are connected to a combined flow recorder. The primary devices for the flow recorder are the discharge weirs in the grit chambers which reflect the respective pump discharge rates. Although the pump station has facilities for automatic operation, it is normally operated on a manual basis. Consequently, it was difficult to determine a meaningful daily flow variation.

The computation of the flow rates and their variations throughout the day was similar to that for Pump Station H. The sum of the results of these computations was compared to the total daily flow normally reported by the operator. These latter values were derived by multiplying daily pump operating times by the average pump discharge rates. The ratio of the average daily rates as computed by the above procedures to those reported was found to be 0.85. This ratio was used to adjust the recorded flow data, particularly when the data were used with measured flow.

Overflow Structure

This overflow structure is located upstream from Pump Station H at Elmhurst Creek near the Oakland - Alameda County Coliseum. The overflow provides a relief for Pump Station H in cases of mechanical failure or flow beyond its capacity. A flow measuring device was desirable to provide a means for measuring the volume of water that overflowed.

The flow measuring arrangement consisted of measuring the liquid levels on both sides of a rectangular tide gate and extracting the flows from the manufacturer's rating curve. Because of difficulties in installation and operation, no usable flow measurements were obtained during periods of overflow. Therefore, estimates of overflow volumes were derived from the Pump Station H operating records, which indicated the time period between starting and stopping of the station together with the flow rate when it was stopped.

SAMPLING AND TESTING

Both wet weather and dry weather samples were taken at most of the locations listed in Table 6. A total of 926 samples were taken, 787 during wet weather and 139 during dry weather. The wet weather samples were of the grab type and the dry weather samples were of the composite type, proportional to time but not flow rate. Eight different laboratory analyses were made on each sample. Therefore, except for those tests in which no results were obtained, approximately 7,400 analyses were made during the investigation. An evaluation of the data is presented in Section 5.

WET WEATHER SAMPLING PROCEDURES

The equipment used in taking wet weather flow samples depended on the location of the metering station. If the station was outside the travelled way, grab samples were obtained by using a bucket suspended on a rope. If it was within the travelled way and traffic conditions rendered manual sampling unsafe, an Edson[1] lever action diaphragm pump was used in conjunction with a previously installed one and one-half inch suction line. A quarter-drum located at curb-side was used to receive the first discharge from the pump so that the sample taken immediately thereafter would reflect the actual condition in the sewer. The samples were collected in one-gallon plastic containers from which a six-ounce sample was extracted for separate coliform analyses. Because of the lapse between the time of sampling and the actual analyses, the six-ounce samples were immediately placed in an insulated chest and covered with ice.

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[1]Mention of commercial products does not imply endorsement by the Federal Water Quality Administration.

Sampling periods occurred during all times of the day and night. One or more station wagons were kept loaded and ready with the necessary equipment in preparation for a storm. The equipment included the pumps, buckets, one-gallon containers, six-ounce containers, ice chests, rubber rain suits, recording equipment, and safety devices. Many trips were made during which no samples were obtained.

The frequency of wet weather sampling also depended on the location of the metering station. The initial sampling cycle was: 15-minute intervals for the first two hours, 30-minute intervals for the next two hours, and one additional sample at the end of the fifth hour. This cycle was modified to suit the location once the nature and characteristics of the contribution from the subarea had been experienced. For example, samples at the water pollution control plant were obtained at one-hour intervals for a period of 48 hours after the beginning of the storm. Samples at Pump Station H were taken at 30-minute intervals over the five-hour period. Sampling periods longer than five hours were also used at Pump Station A and Pump Station H. It was found that 15-minute intervals for the entire four- or five-hour period were required for several of the smaller subareas.

DRY WEATHER SAMPLING PROCEDURES

Two types of portable samplers were used to obtain the dry weather flow samples. Three Hinde Effluent Samplers[2] were used; this type consisted mainly of a positive displacement pump capable of a 20-foot or higher suction lift. It had a 15- and 30-minute selective time cycle, a 12-volt DC battery, and a two-gallon storage container. One Surveyor Automatic Composite Sampler[2] was used, consisting of a rubber vane type pump that could provide only a six-foot suction lift. It also had an adjustable time cycle and operated on a 12-volt DC battery.

The only real difficulty encountered in using the automatic samplers was that the suction tubing was so small that stringy and large size material tended to plug the line. A partial solution was found with a stilling well arrangement made of 20-mesh galvanized wire fabric. The suction tubing was placed inside the screened well which prevented the large materials from reaching the inlet of the tube. It was necessary periodically to clean the accumulation of debris away from the wire screen.

The samplers were operated for a period of time which included two weekends and five week days at each of the metering locations. Because of the plugging of the suction line, the samplers were operated, in most cases, for longer times than the anticipated sampling period.

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[2]Mention of commercial products does not imply endorsement by the Federal Water Quality Administration.

No dry weather flow samples were obtained at Nineteenth Street because the depth of the manhole (24 feet) exceeded the suction lift capability of the samplers.

Samples were picked up daily. After the sample was mixed in the storage container, a portion of the liquid was transferred into a one-gallon container, from which a six-ounce sample was taken for coliform analyses. The samples were stored in ice chests or in refrigerators at approximately 35 to 40 deg F until they were analyzed.

LABORATORY TESTS AND PROCEDURES

All analyses except those for coliform were made in the Metcalf & Eddy laboratory. A majority (546) of the coliform analyses were made by the EBMUD Water Pollution Control Plant Laboratory, and the remainder (380) were made by the Cook Research Laboratories, Inc., Menlo Park, California. All coliform analyses were made by the multiple-tube fermentation technique and were carried only through the presumptive test. The EBMUD Laboratory used five tubes with three or more decimal dilutions, whereas Cook Laboratories used three tubes with three or more decimal dilutions.

The following analytical tests were conducted on the samples:

1. Total solids
2. Volatile suspended solids
3. Suspended solids
4. Settleable solids
5. BOD
6. COD
7. Oil and grease
8. Coliform bacteria

All of the tests made during this investigation were conducted in accordance with Standard Methods for the Examination of Water and Waste Water, 12th edition, published by the American Public Health Association (1).

At the peak of the wet weather sampling, the number of samples collected exceeded the analytical capacity of the Metcalf & Eddy laboratory. During this period the samples were kept in a refrigerator at a temperature of 35 to 40 deg F, or were packed in ice in 55-gallon steel drums until they could be analyzed. Normally, under these conditions, the samples were held no more than two days, although on two occasions they were held as long as four days.

PRECIPITATION GAGING

Recording gages giving at least hourly rainfall amounts were needed to satisfy the requirements for precipitation data. However, in areas where the temporal variation in rainfall is slight, the temporal

distribution from a few recording gages was applied to the rainfall measured in nonrecording (tube) gages. The distribution of recording gages depends mainly on the variation in the temporal distribution of rainfall, whereas the distribution of nonrecording gages depends on the variation in the areal distribution of rainfall. Therefore, it was possible to substitute nonrecording gages in some cases where total daily rainfall was needed to reflect areal distribution only.

For most of the subareas, a recording gage was required within or adjacent to the subarea because the temporal variation of the rainfall was unknown. The variation of the areal distribution of rainfall was known to some degree from a map of mean annual precipitation presently used by the Alameda County Flood Control and Water Conservation District which was established from data covering 30 years of records. The mean annual precipitation, however, being the total rainfall for an average year, did not reflect the actual variation in the areal distribution of rainfall for each storm. Thus, more gages were considered necessary to define adequately the areal distribution of storms and to provide sufficient data to construct a map of areal rainfall distribution.

The locations and types of precipitation gages used are shown on the map in Figure 12. Table 7 lists the name, location, applicable subarea, receiver diameter, type, and location of record for each gage. Official gages are those installed in conformance with U.S. Weather Bureau standards and checked periodically by Weather Bureau personnel. Unofficial gages are the other gages in the area that are maintained by various agencies and individuals; these may or may not be as accurate as the official gages.

For this study, five recording gages were added to the existing gages, bringing the total number of recording gages to 11, or about one for every 7.3 square miles of area. Three nonrecording gages were also added, bringing the total number of gages in the study area to 20, or about one for every 4.0 square miles of area. Data from two additional gages just outside the study area were collected for describing the overall rainfall pattern.

The locations of the additional gages were selected to meet the recommendations of the U.S. Weather Bureau providing: 1) good exposure to the elements; 2) adequate distance from individual high structures and trees which could shed more water onto the gage or create eddy currents; 3) protection from high winds; and 4) security from damage or disruption by vandalism or foot and automobile traffic. The prevailing local conditions or topography made it necessary to place more emphasis on one requirement than another; however, it is considered that the gages were located in positions that presented the best available combination of exposure, security, and correlativity.

SHIRLEY DRIVE
SHEPHERD CANYON
GLENWOOD GLADE
SPRUCE STREET
CALIF SCHOOL FOR THE BLIND
BERKELEY USGS
PIEDMONT
MCMILLAN
CARLETON AVE
OAKLAND CITY HALL
ALAMEDA NAS

FIG. 12 LOCATIONS AND TYPES OF PRECIPITATION GAGES

TABLE 7

LOCATIONS AND TYPES OF PRECIPITATION GAGES USED IN FLOW MEASURING AND SAMPLING PROGRAM

Gage name	Gage location	Applicable subarea	Official or unofficial	Gage description: Type	Gage description: Recording	Gage description: Receiver diameter, inches	Location of record
Berkeley	University of California, Berkeley	[illegible]	Official	--	Yes	--	U.S. Weather Bureau, San Francisco
California School for the Blind*	California School for the Blind, Berkeley	[illegible] Avenue	Unofficial	Tube	No	8	Metcalf & Eddy
Carleton Avenue*	8th and Parker, Berkeley	Carleton Avenue	Unofficial	Weighing	Yes	8	Metcalf & Eddy
Spruce Street*	Shasta Road and Queens Road, Berkeley	Spruce Street (and Glen Street)	Unofficial	Weighing	Yes	6	Metcalf & Eddy
Capistrano Drive*	10701 Runnet, Oakland	Capistrano Drive	Unofficial	Weighing	Yes	8	Metcalf & Eddy
Oakland City Hall*	14th Street and Washington, Oakland	Nineteenth Street	Unofficial	Weighing	Yes	8	Metcalf & Eddy
Pump Station 2*	Frederick Street, Oakland	Pump Station 2 (and High Street)	Unofficial	Weighing	Yes	8	Metcalf & Eddy
Skyline Boulevard*	13170 Skyline Boulevard, Oakland	Skyline Boulevard	Unofficial	Tube	No	8	Metcalf & Eddy
Shepherd Canyon*	5921 Shepherd Canyon, Oakland	Trestle Glen Road	Unofficial	Tube	No	8	Metcalf & Eddy
Piedmont	Vista and Highland Avenue, Piedmont	Trestle Glen Road	Unofficial	--	Yes	--	Alameda County Flood Control District
Oakland International Airport Meteorological Station	Oakland International Airport	Overall Study Area	Official	--	Yes	--	U.S. Weather Bureau, San Francisco
Upper San Leandro Filters	Upper San Leandro Filter Plant	--	Official	--	Yes	--	EBMUD
Alameda Naval Air Station	Naval Air Station, Alameda	--	Unofficial	Weighing	Yes	4	National Weather Records Center, Asheville, North Carolina
Chabot Filters	Chabot Filter Plant	--	Unofficial	Tube	No	8	EBMUD
Chabot Reservoir	Chabot Reservoir	--	Unofficial	Tube	No	8	EBMUD
EBMUD Plant	EBMUD Water Pollution Control Plant	--	Unofficial	Tipping-Bucket	Yes	12	EBMUD Water Pollution Control Plant
Oakland, Glenwood Glade	172 Glenwood Glade, Oakland	--	Unofficial	Tube	No	1/4	U.S. Weather Bureau, San Francisco
Oakland, McMillan	5525 McMillan, Oakland	--	Unofficial	Tube	No	1	U.S. Weather Bureau, San Francisco
Oakland, Shirley Drive	7200 Shirley Drive, Oakland	--	Unofficial	Tube	No	1	U.S. Weather Bureau, San Francisco
Oakland, 39th Avenue	4097 39th Avenue, Oakland	--	Unofficial	Tube	No	4	U.S. Weather Bureau, San Francisco
Orinda Filters**	Orinda Filter Plant	--	Unofficial	--	Yes	--	EBMUD
Upper San Leandro Reservoir**	Upper San Leandro Reservoir	--	Unofficial	Tube	No	8	EBMUD

*Gages installed specifically for this investigation.
**Outside of study area.

The Benvenue Avenue subarea is located close to the Berkeley Campus and in similar terrain. A nonrecording tube gage was located within this subarea at the California School for the Blind. This gage provided the actual total quantities of precipitation in the subarea. The time distribution of that precipitation was derived from the recording gage on the campus.

No existing recording precipitation gages were found within a reasonable distance of the Carleton Avenue subarea. Accordingly, a recording gage was installed adjacent to the administration building at Cutter Laboratories located on Carleton Avenue.

The existing recording precipitation gage nearest to the Spruce Street subarea is located at the University of California at Berkeley. This subarea is sufficiently dissimilar, orographically, from the campus that precipitation in the two areas is not always related; therefore, a recording gage was located at the fire station at the corner of Shasta Road and Queens Road to provide a better description of the rainfall patterns.

The Capistrano Drive subarea is about two miles east of the Metropolitan Oakland International Airport. No existing precipitation gages were available so a recording gage was installed within the subarea and correlated with the official recording gage at the Oakland Airport. The site selected was inside a fenced industrial yard which offered good exposure and relatively good security.

A recording gage was installed at the Oakland City Hall. Located only four blocks from the Nineteenth Street subarea, it was used to develop the record for that subarea.

The Pump Station J and High Street subareas are orographically similar, they receive essentially the same mean annual rainfall, and they are sufficiently close together that it was assumed one recording precipitation gage would adequately describe the rainfall patterns for both subareas. Because no existing gages were available, one recording gage was installed at Pump Station J.

The Skyline Boulevard subarea is in a relatively unique location in the hills above Oakland. No precipitation gages existed close enough to this subarea to provide an accurate description of the rainfall. Because it is a newly developing area which was not expected to allow much infiltration, a nonrecording gage within the immediate vicinity of the intersection of Brookpark Road and Skyline Boulevard was installed. The City of Oakland Fire Station, Engine No. 21, on Skyline Boulevard was selected as the site for the gage because of good security and exposure characteristics.

The City of Piedmont maintains a recording gage at the City Hall, located reasonably near the Trestle Glen Road subarea. Since the terrain and orography near the gage is similar to that of the subarea, the precipitation data were developed from that gage and from a nonrecording gage located at the City of Oakland Corporation Yard on Shepherd Canyon Road.

SECTION 5

ANALYSIS AND EVALUATION OF DATA

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SECTION 5

ANALYSIS AND EVALUATION OF DATA

The data collected during the flow measuring and sampling program are too voluminous to be presented in this volume of the report. In the first part of this section, data from selected measuring stations are introduced by means of typical graphs and tables to illustrate the characteristics of the storm water infiltration. This discussion is followed by a description of the detailed analyses and evaluations used in a search of relationships between rainfall and the quantity and quality of extraneous flow caused by storm water infiltration.

To provide a further definition of the major sources of storm water infiltration, a flow routing computer program was developed and used for simulating the measured hydrographs in the sewer system from various subareas. Much of the data analysis was directed toward developing factors and coefficients necessary for this program. The use of the computer program is described briefly here. The computer program is also used in Section 6 to estimate the overall infiltration quantities in the East Bay sewer systems.

It would have been useful to include quality routing in the computer program, but it was realized early in the investigation that development of a water quality model to that point was beyond the scope of this study. Therefore, quality relationships needed for evaluation of effects of untreated overflows on the Bay were selected from data collected at points along the interceptor that were representative of the major overflows along the interceptor and the bypass at the water pollution control plant. The values for selected water quality relationships are used for evaluating the effects of overflows on San Francisco Bay in Section 9.

Because rainfall analyses usually involve statistical relationships, it was not possible to develop all the desired rainfall characteristics only from the data collected during one season. Historical data were therefore collected from various sources; the findings and analyses of both sets of data are included in this section.

PRESENTATION OF BASIC DATA

FLOW MEASUREMENTS

Diurnal Variations in Dry Weather Flow

The diurnal variations developed as part of this data collection program represent conditions found in the subareas specifically between November 1968 and April 1969. The flow variations are not truly unit

flow variations because they represent data for specific sizes of drainage areas and cannot be directly applied to areas smaller than those in which they were measured.

Table 8 summarizes the areal contributions of dry weather flow for each subarea and lists the land area (acres) involved, the respective land use category, and the diurnal variations. Figure 13 illustrates the diurnal variations for both week days and weekend days for the Pump Station A drainage area. The values presented in Table 8 and Figure 13 are averages of data from up to ten different days, not necessarily consecutive, and not noticeably influenced by preceding wet weather conditions.

Extraneous Flow Hydrographs

Extraneous flow hydrographs were developed by subtracting the appropriate dry weather flow from the measured wet weather flow; both time and type of day (week day versus weekend day) were considered. The area below the resulting extraneous flow hydrograph was measured to provide the volume of extraneous flow for the respective storm. This volume was then compared to the volume of rain falling during the storm. Figure 14 illustrates a typical extraneous flow hydrograph, together with the rainfall histogram for the Pump Station A drainage area.

The preparation of extraneous flow hydrographs for separate sanitary sewers representing infiltration of storm water have not been attempted previously, as indicated by an extensive literature review. Infiltration rates are normally given for separate sewer systems without regard either to time distribution or to effect of rainfall.

QUALITY ANALYSES

Dry Weather Data

As has been mentioned, the samples for dry weather quality data were collected with automatic sewage samplers using either a small positive displacement or a rubber impeller pump with a small diameter suction tube. Although the samples were taken at either 15- or 30-minute intervals and composited throughout the day, the results of the analyses were somewhat erratic. Table 9 shows a summary of the dry weather quality data consisting of average, minimum, and maximum values compiled from both week day and weekend day data. These samples were taken between March and May 1969 at periods of time least influenced by preceding rainfall. The data normally represent five samples collected over a 24-hour period at each subarea.

Wet Weather Data

It was concluded that the best way to illustrate the variations of quality during wet weather conditions was to plot all of the quality parameters together with rainfall and extraneous flow versus time on

TABLE 8

AREAL CONTRIBUTIONS OF DRY WEATHER FLOW FOR EAST BAY DRAINAGE AREAS, November-April, 1968-69

Area description	Land area, acres	Land use*	Average areal flow contribution, gad**: Week days	Average areal flow contribution, gad**: Weekend days	Diurnal variations, Percent of average flow contribution: Week days Min.	Week days Max.	Weekend days Min.	Weekend days Max.
Suburban								
Seminary Avenue	81	MD RES	3,500	3,795	57	140	66	139
Carlton Avenue	26	INDUS	5,930	1,785	63	153	88	113
Glen Street	63	LD RES	1,440	1,310	72	130	67	134
Spruce Street	459	LD RES	1,480	1,525	64	156	58	144
Capistrano Drive	95	LD RES	2,840	2,630	67	129	57	156
Nineteenth Street	41	HD RES + COMM	5,090	4,590	23	185	26	181
Pump Station I	64	INDUS	3,855	2,540	54	147	72	128
Skyline Boulevard	82	LD RES	975	375	28	193	26	173
Trestle Glen Road	645	LD RES	2,215	2,185	55	151	49	147
Major Drainage Areas								
Pump Station A	709	MIXED	1,495	1,735	35	156	41	171
Pump Station H	12,299	MIXED	1,420	1,155	45	145	51	135
Total Drainage Area								
EBMUD Special District No. 1	53,440	MIXED	1,530	1,470	65	140	56	143

*HD RES = High density residential; MD RES = Medium density residential; LD RES = Low density residential; COMM = Commercial; INDUS = Industrial.

**Gallons per acre per day.

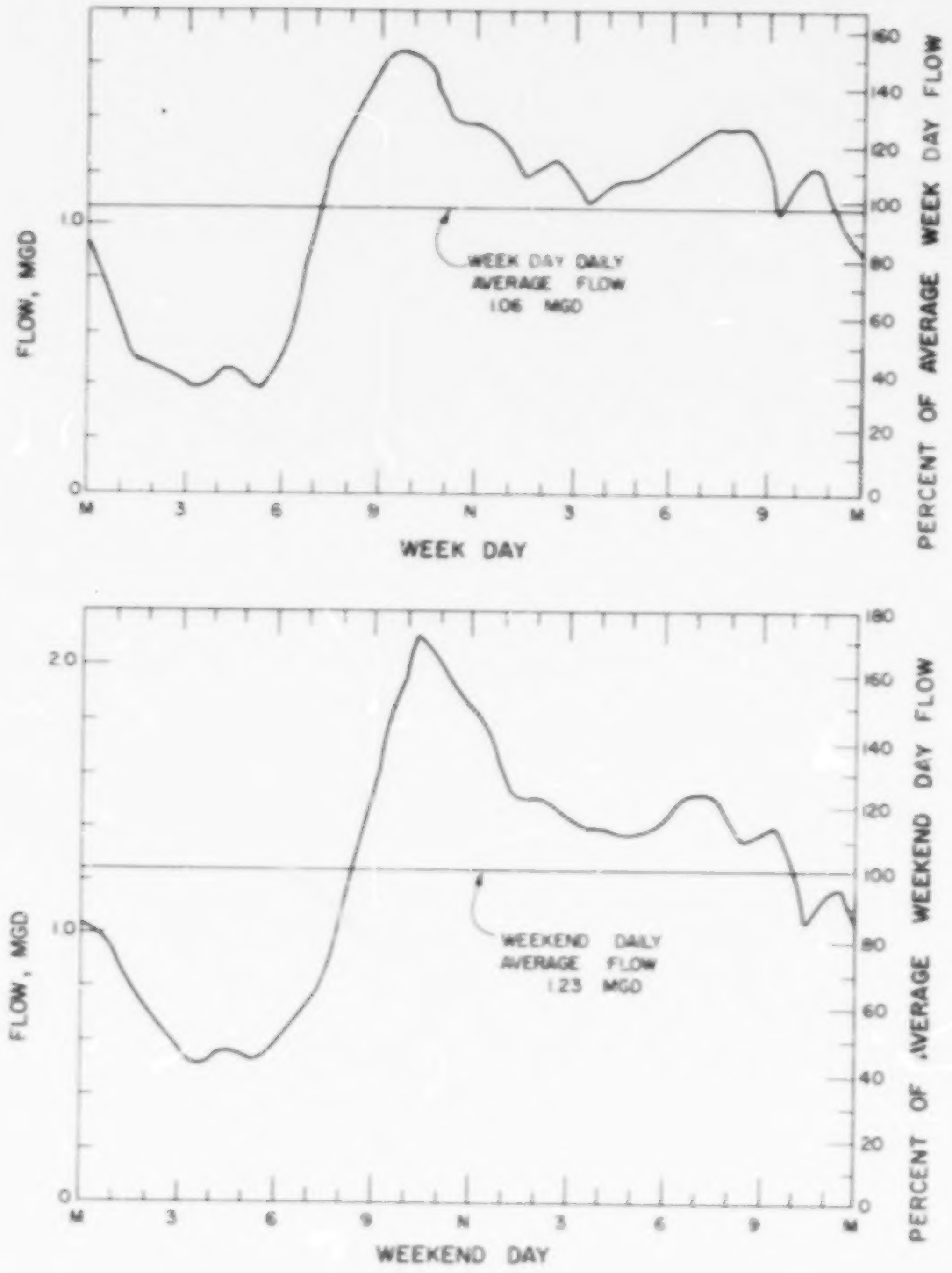

FIG. 13 DIURNAL FLOW VARIATION, PUMP STATION A, NOVEMBER-APRIL 1968-69

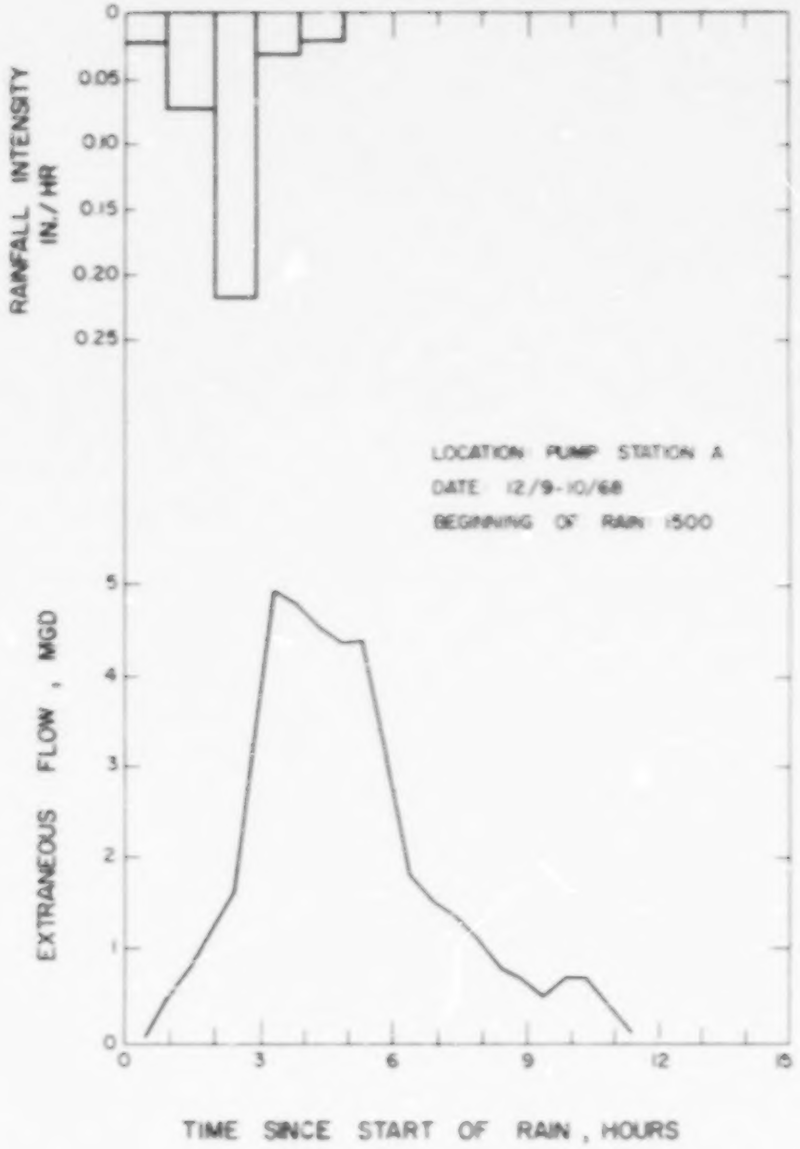

FIG. 14 EXTRANEOUS FLOW HYDROGRAPH AND RAINFALL HISTOGRAM, PUMP STATION A

TABLE 9

SUMMARY OF DRY WEATHER QUALITY DATA FOR EAST BAY DRAINAGE AREAS, MARCH-MAY 1969

Area description	Total solids, mg/l	Suspended solids, mg/l	Volatile solids, mg/l	Settleable solids, ml/l	BOD, mg/l	COD, mg/l	Oil and grease, mg/l	Coliform,* MPN×[illegible]/ml
Subareas								
Bellevue Avenue	344	125	88	3.3	89	266	25.0	643
	256	46	26	0.0	32	175	4.8	230
	520	266	204	10.0	162	400	52.8	1,400
Carleton Avenue	542	199	125	4.6	1,002	1,448	11.7	6,530
	221	55	26	0.2	562	240	4.9	43
	864	536	425	21.5	1,770	1,736	25.9	11,000
Glen Street	474	137	61	2.1	61	205	18.2	789
	410	51	31	0.0	27	128	7.7	43
	662	337	236	21.3	140	480	38.5	4,600
Spruce Street	751	283	182	19.9	126	422	28.8	528
	444	105	50	0.6	30	167	8.1	43
	1,030	428	411	55.7	940	672	51.1	2,400
Capistrano Drive	499	54	37	0.3	315	426	18.3	2,470
	380	31	33	0.0	125	278	16.6	93
	670	117	96	0.5	1,300*	734	77.6	11,000
Pump Station J	752	91	63	0.1	198	367	33.2	1,500
	353	34	22	0.0	117	206	16.2	150
	1,149	192	134	1.8	890	769	170.1	4,600
Skyline Boulevard	406	59	46	1.7	144	500	35.1	[illegible]
	359	18	25	0.0	97	221	20.7	230
	528	131	108	7.6	1,310*	404	58.2	2,400
Trestle Glen Road	637	163	122	1.9	135	267	38.0	1,520
	351	15	39	0.0	98	160	19.9	250
	794	613	406	21.2	280	508	278.1*	4,600
Major Drainage Areas								
Pump Station A	587	108	60	0.5	120	488	30.0	1,230
	381	44	40	0.0	121	275	17.9	430
	1,299*	192	175	4.0	1,000	1,127	92.8	13,000
Pump Station G	680	110	79	0.6	294	340	41.0	1,400
	503	47	58	0.0	112	268	26.7	430
	1,166	205	141	4.0	450	743	53.3	11,000

*Excessively high values disregarded in average values.
Note: The top values are averages of all data, the second values are minimums, and the bottom values are maximums.

a single graph. The rainfall/flow/quality graph for the Pump Station A drainage area, shown on Figure 15, is a typical illustration of the variations that were found.

Because it was difficult to begin sampling at the best time so far as flow was concerned, many of the graphs show quality values during various periods of the extraneous flow hydrograph. Some are on the rising limb portion, some cover the peak, and others cover the recession limb portion of the hydrograph.

RAINFALL GAGING

To illustrate the frequency and nature of the rainstorms during the two peak months of the rainy season, January and February 1969, the corresponding graphs of rainfall at the Oakland Airport are shown in Appendix II. Certain historical rainfall data were collected and analyzed but were not presented in the form of basic data.

ESTIMATE OF RELATIONSHIPS BETWEEN INFILTRATION AND RAINFALL

The sources of storm water infiltration can be described by two general terms: percolation, which represents the entry of vadose water or groundwater through poor pipe walls and joints; and direct connections, which represent the entry of water by any of several sources, such as roof leaders and catchbasins. The sewers in the East Bay Area are subject to infiltration from both sources. As previously mentioned, most of the sewers are old and in poor condition. Also, the maintenance personnel indicated that direct connections are extensive and must contribute substantially to the infiltration problem. The methods used to define gross infiltration characteristics and the proportion of infiltration applicable to each source are described in the following discussion.

GROSS INFILTRATION RATIOS

The study area was divided into five types of land use, as shown on Appendix Figure I-2. The subareas that were selected contained a single type of land use and represented four of the five land use categories: 1) high density residential and commercial, considered as one because they could not be distinguished; 2) medium density residential; 3) low density residential; and 4) industrial. The fifth category (cemeteries, parks, military holdings, and undeveloped areas) does not contribute to dry weather sewage flow. Each of the subareas was further classified according to its topography (hilly or flat) and age of sewer system (old or new). Table 10 lists each subarea with its respective classification. During the flow measuring program, extraneous flow volumes were measured for several storms for each of the subareas as well as for the two control areas, Pump Station A and Pump Station B.

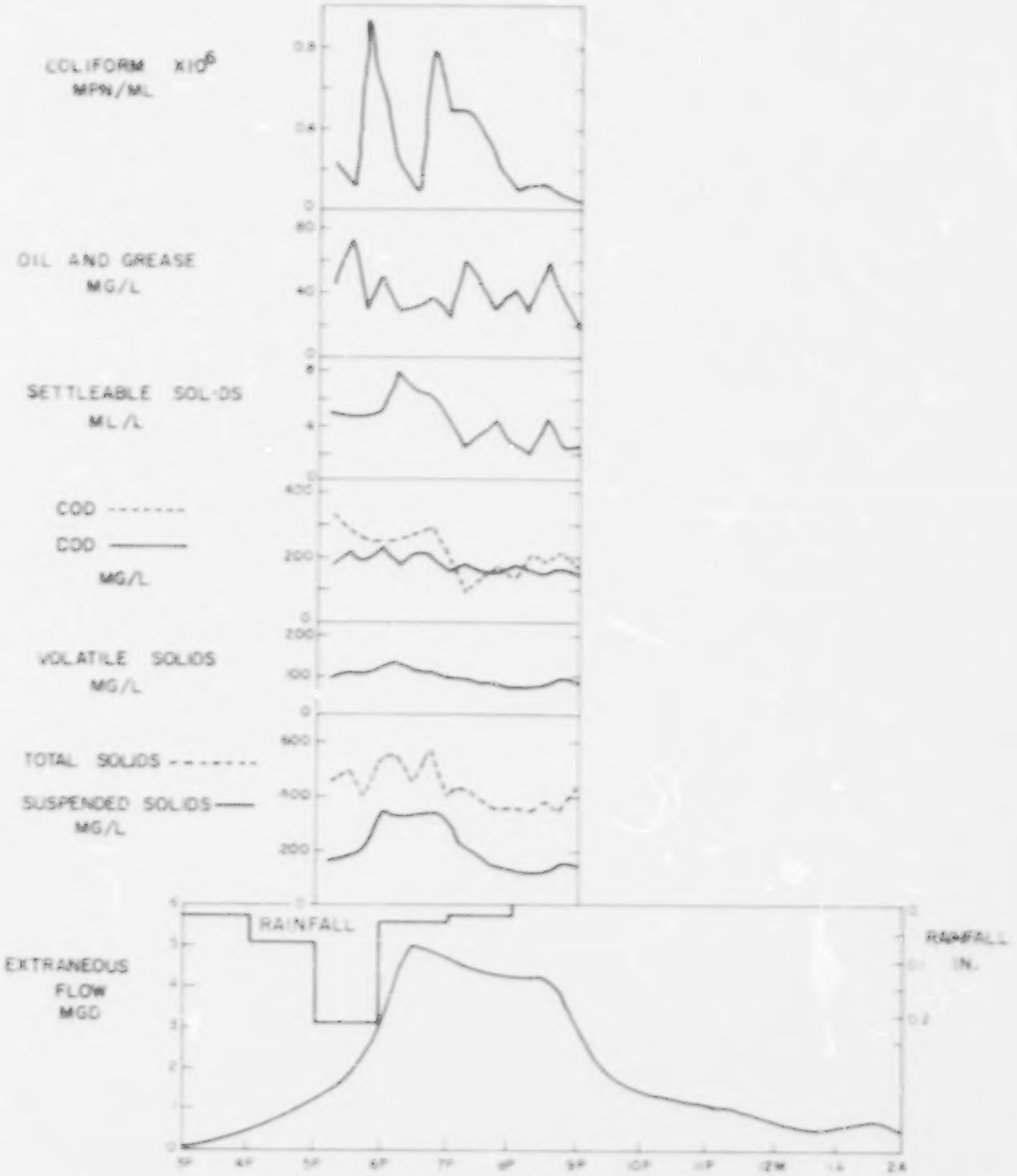

FIG 15 RAINFALL/FLOW/QUALITY GRAPH,
PUMP STATION A, DECEMBER 9-10, 1968

TABLE 10

*GROSS INFILTRATION RATIOS FOR EAST BAY DRAINAGE AREAS**
1968-69

Area description**	Land use topography and sewer age classification***	Ratio
<u>Subareas</u>		
Spruce Street	LD RES, Hilly, Old Sewers	0.03
Skyline Boulevard	LD RES, Hilly, New Sewers	0.01
Capistrano Drive	LD RES, Flat, Old Sewers	0.11
Benvenue Avenue	MD RES, Flat, Old Sewers	0.12
Nineteenth Street	HD RES and COMM, Flat, Old Sewers	0.14
Pump Station J	INDUS, Flat, Old Sewers	0.08
Carleton Avenue	INDUS, Flat, Old Sewers	0.06
Trestle Glen Road	LD RES, Hilly, Old Sewers	0.07
<u>Major Drainage Areas</u>		
Pump Station A	MIXED	0.246
Pump Station H	MIXED	0.08

*Gross infiltration ratio is $\frac{\text{volume of extraneous flow}}{\text{volume of rainfall}}$.

**For more detailed description, see Table 5.

***HD RES = High density residential; MD RES = Medium density residential; LD RES = Low density residential; COMM = Commercial; INDUS = Industrial.

Table 10 does not list a flat, residential area that has new sewers because there are none within the study area. The flat and most easily subdivided land was developed prior to 1960.

The volumes of storm water infiltration, defined as extraneous flow, were plotted against the volume of rainfall for each subarea and control area. The plots of these points are shown in Appendix Figure II-3. The points did not plot in straight line positions, but a line of best fit was drawn visually through the points. Because these plots represent the first attempt to define the infiltration ratios, the lines were drawn through the origin of the axes. Technically, these lines should intercept the vertical axes at some point above the origin because some rain is required to initiate infiltration. The volume of rain required was unknown, but it was assumed that the slope of the line would not be greatly affected. The ratio of extraneous flow to rainfall volume, computed from the slope of the line, was termed the gross infiltration ratio. Table 10 lists the gross infiltration ratios for each subarea.

In most of the subareas, there was a substantial variation in the amount of storm water that entered the sewer. Significant fluctuations in base flow during periods of rainfall may account for some of the differences in apparent volumes. The extraneous flow hydrograph was developed from the total hydrograph by subtracting the average dry weather flow for the respective time of day.

Two other factors which may account for the variation in infiltration ratios are the rainfall intensity and duration and the antecedent dry period, although neither could be confirmed. Data from five subareas were investigated in an effort to find a definable relationship between the antecedent dry period and the infiltration ratio, but none was found. Similarly, data from four subareas were investigated in an effort to determine the existence of a relationship between rainfall and infiltration versus antecedent dry period. Again, no relationship was found.

The normal units used to describe infiltration rates into sewers are gallons per inch of diameter per mile per day. To provide a means of comparing the results obtained herein with those published elsewhere, similar values were computed for two subareas. Because infiltration during a rainstorm varies with time, some averaging of values was necessary.

The first infiltration rates were computed for the Capistrano Drive subarea, which contains 29.62 inch-diameter-miles of pipe, exclusive of house connections. Assuming that each house is served by a four-inch sewer and that the distance from the house to the main sewer is 50 feet, the 570 houses in this subarea add 21.6 inch-diameter-miles of pipe resulting in a total of 51.2 inch-diameter-miles of pipe. The maximum extraneous flow measured for the storm on February 15, 1969, was 0.87 mgd. This flow is therefore equivalent to an infiltration rate of 17,000 gallons per inch of diameter per mile per day. This infiltration

rate is also equivalent to 9,150 gad. Considering the total 1968-69 season, the extraneous flow measured 1.7 MG during a period of 224 hours. The average storm water infiltration rate was therefore 3,600 gallons per inch of diameter per mile per day or 1,900 gad.

A similar analysis was made for the Skyline Boulevard subarea because it represents an area of new sewers. Having 24.9 inch-diameter-miles of sewer including house connections, the maximum infiltration rate was 5,210 gallons per inch of diameter per mile per day for the storm on February 15, 1969. The seasonal average was found to be 1,200 gallons per inch of diameter per mile per day. The areal contributions were 1,600 and 360 gad, maximum and average, respectively.

Another way to describe infiltration rates is to compare the maximum flow rate during wet weather to the dry weather flow, either for the same time of day or for the average daily flow. Table 11 gives typical ratios of wet weather flow to dry weather flow for each subarea.

Further investigation and analyses of these and other data will be necessary before all of the significant parameters affecting infiltration are known. Seemingly, the duration of a storm should be considered with the antecedent dry period, since infiltration is expected to increase as the antecedent dry period decreases. Perhaps the relationship using the hydraulic parameter of antecedent precipitation index could be found with the aid of coaxial plots to relate other significant parameters. Such an analysis requires more data than were gathered for this study.

RELATIVE MAGNITUDE OF THE TWO SOURCES OF STORM WATER INFILTRATION

Neither the sources nor the magnitude of storm water infiltration in sanitary sewers was known prior to the data collection and development phase of this study. Examination of the extraneous flow hydrographs indicated that the source of infiltration contributed flow almost immediately after the start of a storm. On the other hand, the examination indicated that the time of concentration was quite long (several hours), as measured from the beginning of the storm to the peak of the hydrograph. This seemed to confirm the previous conclusion that two general sources of infiltration existed, percolation and direct connections. To determine the relative magnitude of infiltration from each source, it was decided to model the hydraulic response of the pipe system to infiltration by use of a flow routing computer program. The advantage of such a program was that the hydrographs from the two major components of infiltration could be computed separately and compared.

Selection of Flow Routing Program

Several methods for routing flow through sewers were investigated: the Corps of Engineers program using the Muskingum method; the Chicago method; the Los Angeles method; and the Road Research Laboratory method (RRL Method). Each of these methods recognizes the available sewer

TABLE 11

RATIO OF WET WEATHER FLOWS TO DRY WEATHER FLOWS
1968-69

Area description*	Ratio of peak wet weather flows to: Average daily dry weather flow	Average flow for the same time of day
Subareas		
Spruce Street	5.2	3.8
Skyline Boulevard	4.0	3.8
Capistrano Drive	5.1	4.1
Benvenue Avenue	5.1	4.8
Nineteenth Street	8.1	5.2
Pump Station J	4.5	3.2
Carleton Avenue	5.[illegible]	6.1
Trestle Glen Road	2.1	2.3
Major Drainage Areas		
Pump Station A	9.1	6.4
Pump Station H	4.0	3.0
EBMUD Plant	4.3	3.7

*For more detailed description, see Table 5.

volume as a means of providing storage and thereby dampening the hydrograph. The final selection was the RRL Method because of its relative simplicity, the availability of the basic program, and an indication of successful use based upon published reports.

A sample form of the RRL program, obtained directly from the Road Research Laboratory in England, was modified to make use of the data that had been collected and developed during this investigation. The final program operated under the assumption that the flow in the sewer was open channel and could be described by Manning's equation.[3] The program routed the flow, allowing for storage in the pipe in a manner similar to that for routing through a reservoir. It simulated for each pipe the time distribution and the quantity of dry weather sewage flow by each type of land use. It also simulated the addition of storm water infiltration in each pipe length based upon its respective land use.

Initially, the routing program used the gross infiltration ratios developed previously to simulate storm water infiltration. The simplicity of these ratios aided in the development of the program. However, the objective was to define the infiltration ratio in terms of the two sources: percolation and direct connections. Therefore, the program was later rewritten to accommodate both ratios simultaneously.

The dry weather sewage flow was simulated by considering areal contributions of flow for each land use and for either week days or weekend days, corrected for diurnal variations by using factors in terms of average flow for 15-minute intervals throughout the day. The areal contribution values for week days were identical to those presented in Table 6. The 15-minute interval factors were derived in the program as needed by interpolation from the measured 30-minute interval factors.

Development of Flow Routing Program

The flow routing program was first used for the Capistrano Drive subarea, an older residential area with relatively flat terrain. The first step was to evaluate the sensitivity of the program, i.e., could a coarse pipe network representing combinations of some of the pipes be used in lieu of a fine or detailed network which included all or nearly all of the pipes? Thus, two pipe systems were described: one for a coarse network having an average drainage area of 1.6 acres and another for a fine network having an average area of 10.6 acres. The difference between the computed hydrographs was negligible; therefore, all successive runs were made using the coarse pipe network.

A routing interval of 15 minutes was chosen on the basis of earlier studies of parameter sensitivity which were made during the program

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[3] $Q = A\frac{1.486}{n}R^{2/3}S^{1/2}$.

development. An examination of the actual rainfall histograms and the recorded extraneous flow hydrographs for three storms indicated that a time of concentration of approximately 200 minutes would be in order. Therefore, an inlet time of 180 minutes was selected, allowing for travel time in the system of about 20 minutes.

The second step was to evaluate the use of a single value for infiltration ratio in simulating the measured extraneous flow hydrograph. Constant values for gross infiltration ratio were used for each of three storms, but the computed hydrographs were found to be flatter near the peak flow than the observed hydrographs. Also, the rising limb of the computed hydrographs did not compare favorably with the rising limb of the observed hydrographs. A gross infiltration ratio that varied as the sum of the volumes of preceding rainfall was then developed. The data for the Capistrano Drive subarea were plotted on semilogarithmic paper, and a line of best fit was drawn through the points. An equation of this line was inserted into the program in lieu of the single ratio used previously. The results using this equation produced an excessively high peak and an extended recession curve. Because the use of infiltration ratios representing only a single source of infiltration had failed to simulate the measured hydrographs satisfactorily, it was taken as further evidence that at least a two-phase expression was needed to describe the infiltration phenomenon.

The third step was to rewrite the computer program to permit use of two infiltration ratios: percolation and direct connections. A 180-minute time of entry was used for infiltration involving percolation because of the measured time of concentration from beginning of rainfall. A different time of entry for infiltration ratios involving direct connections was used, based on storm runoff computations which use an inlet time of 2 to 15 minutes. The time of entry for direct connections was selected as 15 minutes because the routing interval used in the program was 15 minutes (hence the program would not recognize shorter periods of time).

In the development of a program which would simulate an observed hydrograph, an allowance had to be made for the rainfall which contributed to saturating the soil and thus was available for infiltration. The rainfall which occurs initially will be absorbed into the soil. If this rainfall is not subtracted in the infiltration analysis, the resulting computed hydrograph will respond with too much flow after the start of the rain.

Application of Flow Routing Program

After modifying the flow routing program to permit the use of two different infiltration ratios, one for percolation and the other for direct connections, the next step was to determine the relative contribution by each of these two ratios. This was accomplished by

running the program for one of the infiltration elements and then adding the effect of the other. The Capistrano Drive subarea, which had a gross infiltration ratio of 0.11, was selected for initial investigation. The results that gave the best fit with the observed hydrograph were composed of the following:

Percolation infiltration ratio	0.09
Direct connection infiltration ratio	0.02
Gross infiltration ratio	0.11

The 0.02 infiltration ratio for direct connections represented a volume of storm water equal to the volume that would be collected from an area equivalent to 10 percent of the roof area--in other words, equal to 10 percent of the roof drains in the study area connected to the sanitary sewer system. The program results using the 0.09 percolation infiltration ratio indicating the results of deducting the first 0.05 inch of rain are shown in the upper graph of Figure 16. The effect of adding the 0.02 infiltration ratio for direct connections is shown in the lower graph of Figure 16. The combined curve as compared to the observed hydrograph is shown in Figure 17. Similar runs were made for two other storms which confirmed the validity of the two ratios.

Three other subareas were investigated using the flow routing program and the same arrangement of infiltration ratios: Spruce Street, Pump Station 3, and Benvenue Avenue. In each subarea the relationship between the two types of infiltration was varied in an attempt to arrive at the proper one for each type of land use. The program was run approximately 50 times before reasonable results were found for these four subareas. The exact duplication of observed hydrographs was not achieved in all cases because of the many variables involved. No doubt, other variables affect infiltration characteristics, but it was concluded that they are either relatively insignificant or are reflected to some degree in the ratios finally selected for use. The infiltration values selected indicate that in the four subareas checked, the major source of infiltration is from percolation and the minor source is from direct connections.

<u>Results of Flow Routing Program</u>

The results of the several computer runs made for each of these four subareas were used to determine the relative magnitude of the two sources of infiltration for the respective land uses they represent. The ratios thus derived are presented in Table 12. The first six subareas listed in the table were used later in computing hydrographs from a control area having multiple land uses. The remaining subareas were used to check the results of the other subareas having the same types of land uses.

The Spruce Street and Trestle Glen Road subareas were expected to have the same infiltration ratios because they represent the same type of

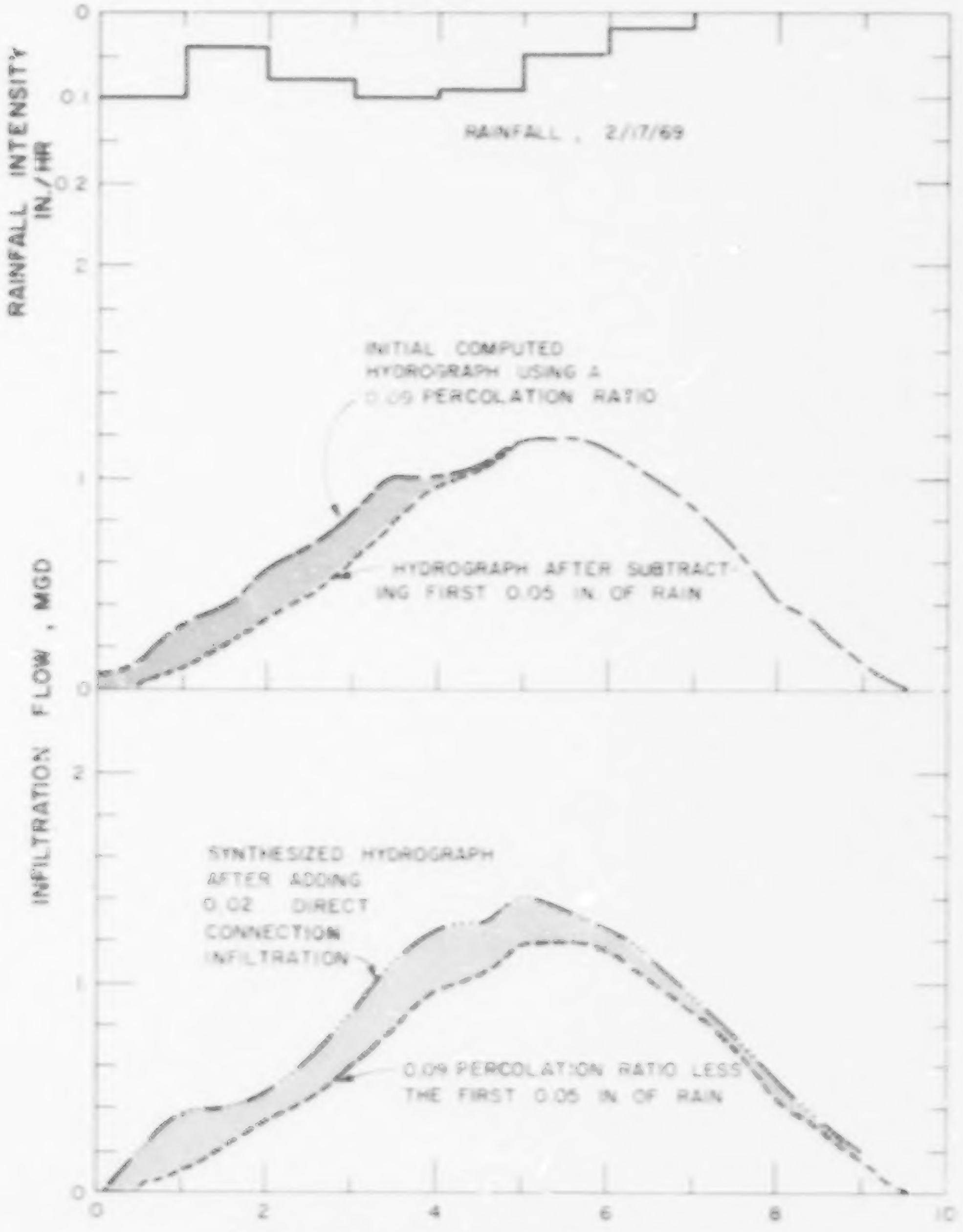

FIG. 16 SYNTHESIS OF EXTRANEOUS FLOW HYDROGRAPH USING PERCOLATION AND DIRECT CONNECTION INFILTRATION RATIOS, CAPISTRANO DRIVE SUBAREA

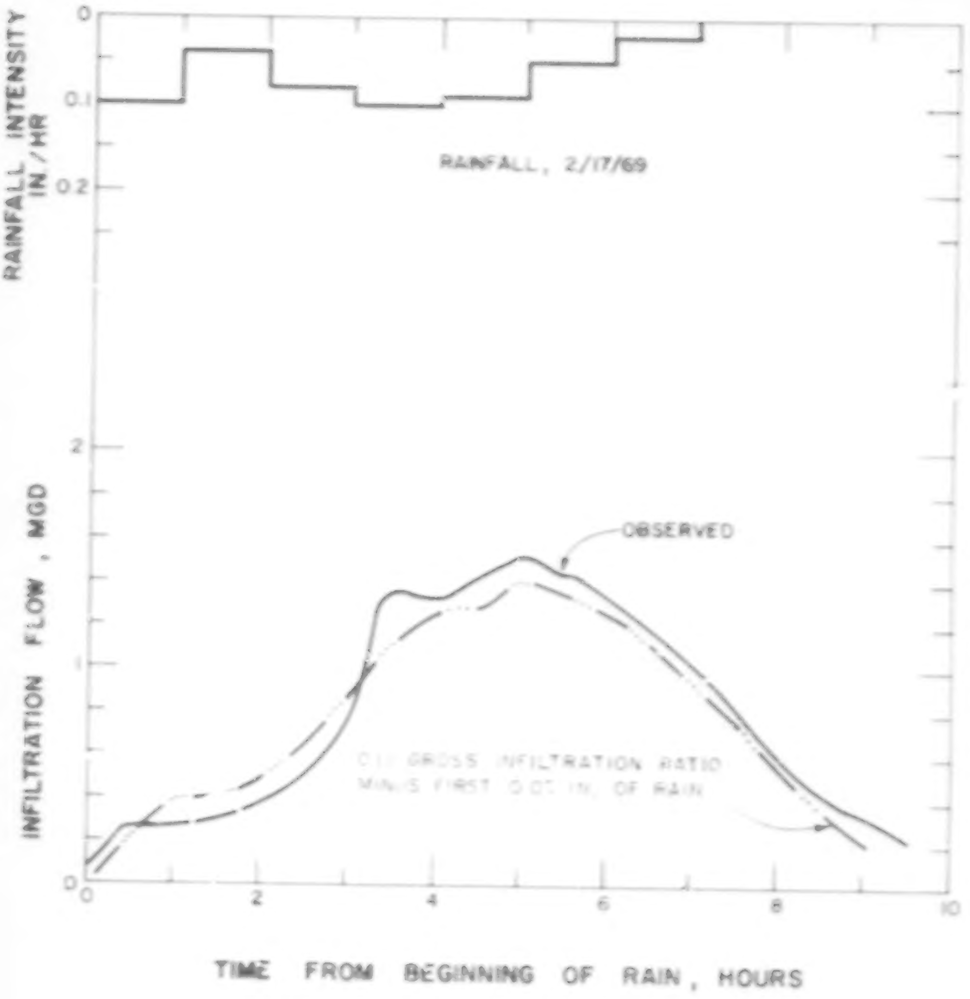

FIG. 17 COMPARISON OF OBSERVED AND COMPUTED EXTRANEOUS FLOW HYDROGRAPHS, CAPISTRANO DRIVE SUBAREA

TABLE 32

INFILTRATION RATIOS FOR PERCOLATION AND DIRECT CONNECTIONS, EAST BAY DRAINAGE AREAS, 1968-69

Area description	Land use, topography, and sewer age classification*	Percolation infiltration ratio	Direct connection infiltration ratio	Gross infiltration ratio
Subareas				
Spruce Street	LD RES, Hilly, Old Sewers	0.01	0.02	0.03
Skyline Boulevard	LD RES, Hilly, New Sewers	0	0.01	0.01
Capistrano Drive	LD RES, Flat, Old Sewers	0.09	0.02	0.11
Benvenue Avenue	MD RES, Flat, Old Sewers	0.10	0.02	0.12
Nineteenth Street	HD RES and COMM, Flat, Old Sewers	0.04	0.10	0.14
Pump Station J	INDUS, Flat, Old Sewers	0.03	0.05	0.08
Carleton Avenue	INDUS, Flat, Old Sewers	0.04	0.02	0.06
Trestle Glen Road	LD RES, Hilly, Old Sewers	0.05	0.02	0.07
Major Drainage Areas				
Pump Station A	MIXED	--	--	0.240
Pump Station B	MIXED	--	--	0.08

*HD RES = High density residential; MD RES = Medium density residential; LD RES = Low density residential; COMM = Commercial; INDUS = Industrial.

land use and approximately the same sewer conditions. However, the data available to establish the infiltration ratio for the Trestle Glen Road subarea were limited to three points, whereas the data for the Spruce Street subarea included eight points. Therefore, a weighted average of these two ratios was later used to represent the similar conditions of land use, terrain, and sewer age. Because of the poor condition of its sewers, the Spruce Street subarea was expected to contribute greater amounts of infiltration than were actually measured. Substantial amounts of extraneous flow may have been discharged to storm sewers upstream from the gaging station which would therefore indicate smaller quantities of infiltration. Overflows are known to exist upstream from the point of measurement, but their extent and nature are unknown.

EVALUATION OF QUALITY DATA

The evaluation of the quality data collected during this investigation was directed toward two principal objectives:

1. To characterize the variations in quality during storm water infiltration.

2. To determine the average values for certain water quality parameters applicable to overflows and plant bypasses.

CHARACTERIZATION OF QUALITY DURING STORM WATER INFILTRATION

The wet weather quality data from several sampling stations were graphically analyzed to determine if any relationships between the various quality parameters and the time from the beginning of the rainfall could be found. The relationship between flow and time after the beginning of rainfall has been previously established; therefore, comparisons were also made of each parameter to the variations and flow rate. These analyses were made for data collected at the Benvenue Avenue and Capistrano Drive subareas and at the Pump Station A and B drainage areas. The following nine parameters or ratios of parameters were investigated:

1. Extraneous flow
2. Total solids
3. Volatile suspended solids
4. Suspended solids
5. Suspended solids/total solids
6. BOD
7. COD
8. Oil and grease
9. Volatile suspended solids/suspended solids.

Plots were prepared for each parameter versus time using all of the data available for each location. Time was measured from the beginning

of rainfall. To aid in comparing the variations in quality with the variations in flow, the time scale was shifted slightly so that the rising limbs of the extraneous flow hydrographs were superimposed on each other. Figure 18 shows the plots for the Benvenue Avenue sampling station. These are similar to the plots made for each station, and the various curves typify the results that were observed.

Although two or more sets of data were available for each station and all of the data were plotted on graphs such as Figure 18, distinct trends were difficult to define. After reviewing all of the data, general trends could be detected.

Figure 19 shows a series of unitless graphs with lines depicting the trends that seemed apparent for each parameter. The plots are intended to represent the parameter concentrations as they vary with time after the beginning of a substantial rainstorm. While these are general trends for the subareas investigated, it should be realized that individual subareas may show considerable variation from storm to storm.

Graphs were also prepared for total solids, BOD, and oil and grease versus extraneous flow for each of the four selected locations (not shown). Time after the beginning of rainfall was ignored in these graphs. There were no noticeable relationships between any of these parameters and extraneous flow.

DETERMINATION OF QUALITY PARAMETERS APPLICABLE TO OVERFLOWS AND PLANT BYPASSES

[illegible] as necessary to determine the expected quality of overflows and plant bypasses by averaging samples taken in the vicinity of overflows and bypasses. These results will be used in Section 9 to evaluate the effects of overflows on the near-shore waters of San Francisco Bay.

Upon reviewing the water quality data from Pump Station A and Pump Station H, the two extreme ends of the interceptor sewer system, a substantial difference in concentrations of water quality parameters was noted. For this reason, it was decided to use the average water quality data from Pump Station A to represent the quality of overflows occurring along the interceptor north of the water pollution control plant and that from Pump Station H to represent the quality of overflows occurring south of the plant. The quality of water bypassing the plant was represented by average data taken at the entrance to the plant during periods of rainfall. Values for coliform bacteria at the plant were not used because both prechlorination and postchlorination are practiced, and the values obtained are of questionable

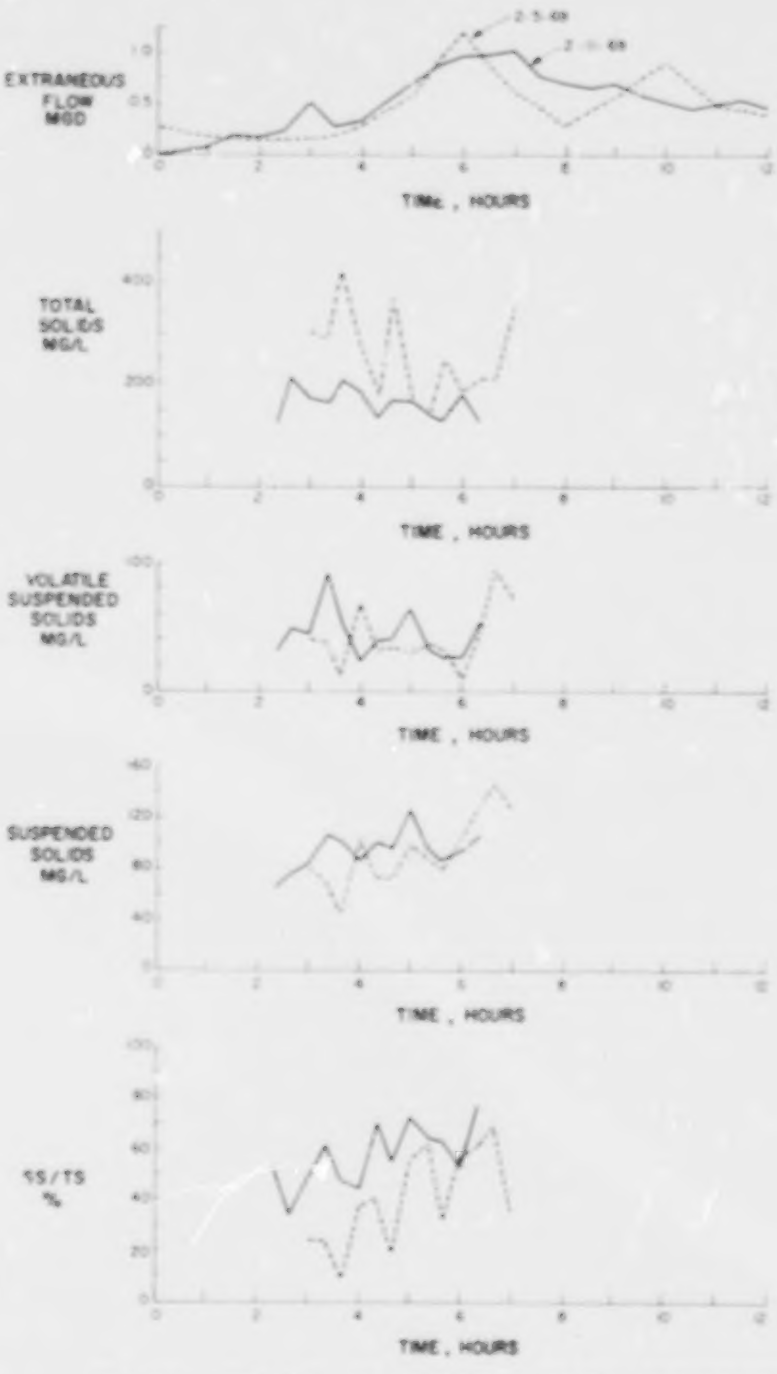

FIG. 18 GRAPHS OF WATER QUALITY PARAMETERS VERSUS TIME DURING WET WEATHER, BENVENUE AVENUE SUBAREA

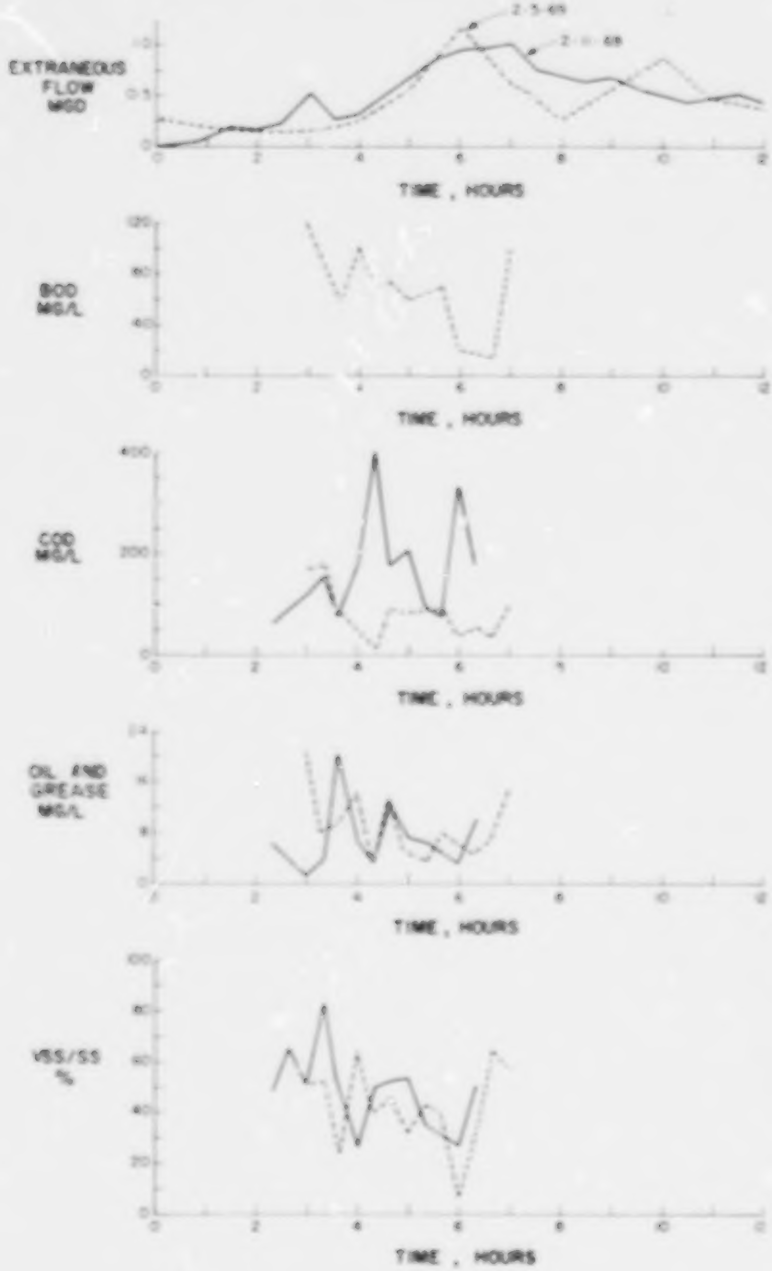

FIG. 18 (CONCLUDED)

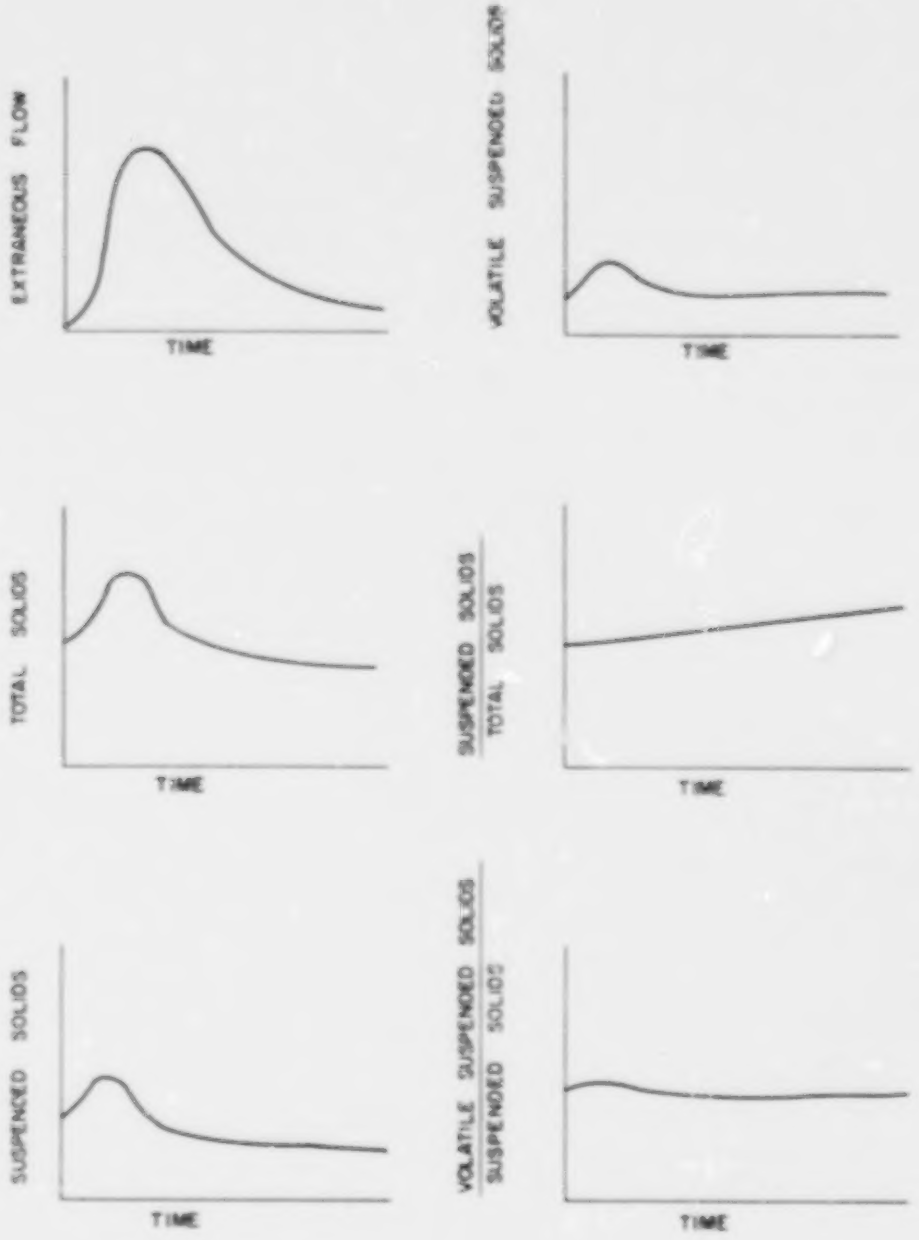

FIG. 19 GRAPHS OF WATER QUALITY PARAMETER TRENDS DURING WET WEATHER

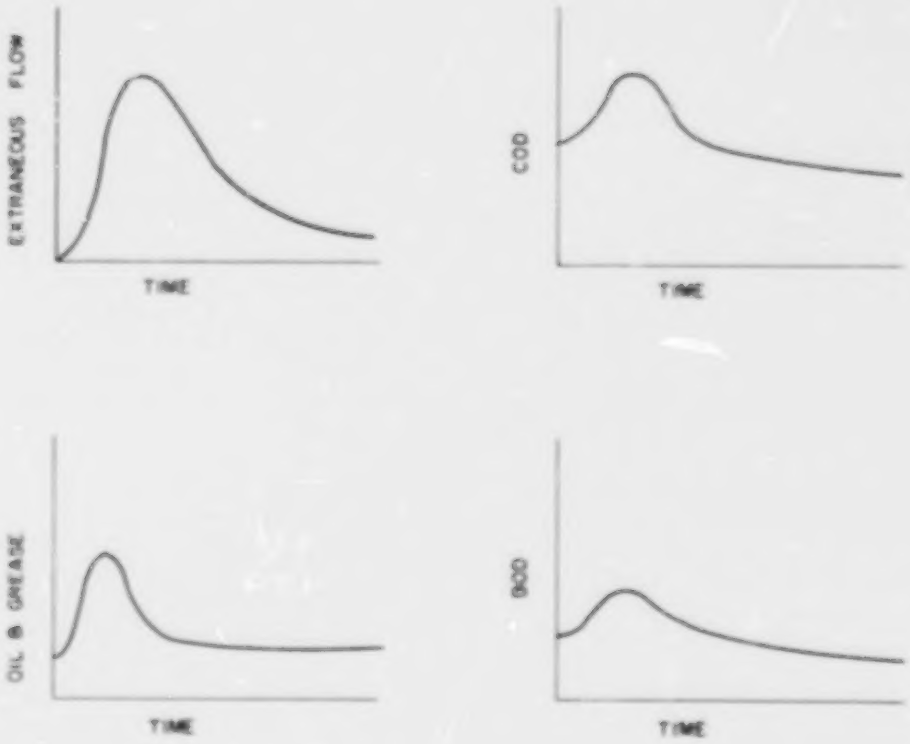

FIG. 19 (CONCLUDED)

significance. The values that were used to evaluate the effects of overflows on the Bay were:

Location	BOD, mg/L	Grease, mg/L	Coliform bacteria, MPN/100 ml
South interceptor	370	105	3.7×10^8
North interceptor	105	21	1.5×10^7
Water pollution control plant	135	40	--

ANALYSIS OF RAINFALL DATA

FREQUENCY ANALYSIS

In order to relate the rainfall data collected during the 1968-69 season to historical storm events, a statistical analysis was made of data from the Oakland Airport for periods of maximum storm rainfall occurring between 1948 and 1968. A family of curves showing the relationship between maximum intensity of rainfall and frequency of occurrence for various values of duration was plotted on Gumbel extreme probability paper. A graph of intensity versus duration for various frequencies of storms was plotted from values extracted from the Gumbel plot and is shown on Figure 20.

A statistical analysis was also made of only the 1968-69 rainfall data from the Oakland Airport, and a family of intensity-duration-probability curves was developed. The curves represent data from the 60 storm events that occurred between November 1968 and April 1969. A storm event was defined as a block of rainfall equal to or greater than 0.02 inch, separated from another storm event by at least five hours. A graph of intensity versus duration for various probabilities of occurrence was plotted from values extracted from this analysis and is shown in Figure 21. The families of curves presented in Figures 20 and 21 are not comparable because the historical data represent annual extreme events, whereas the 1968-69 data represent all events for the season. Figure 21 does serve, however, to characterize the duration and intensity of the several storms during the season.

As a means of visualizing the relationship between the storm events of the 1968-69 season and the historical storm events, a comparison was made between the maximum event of the past season and the statistical analysis of historical data. The results are shown graphically in Figure 22. The comparison shows that the maximum storm event last season had a return period of 1.9 years, that is, the maximum event which took place is expected to be equalled or exceeded once every 1.9 years.

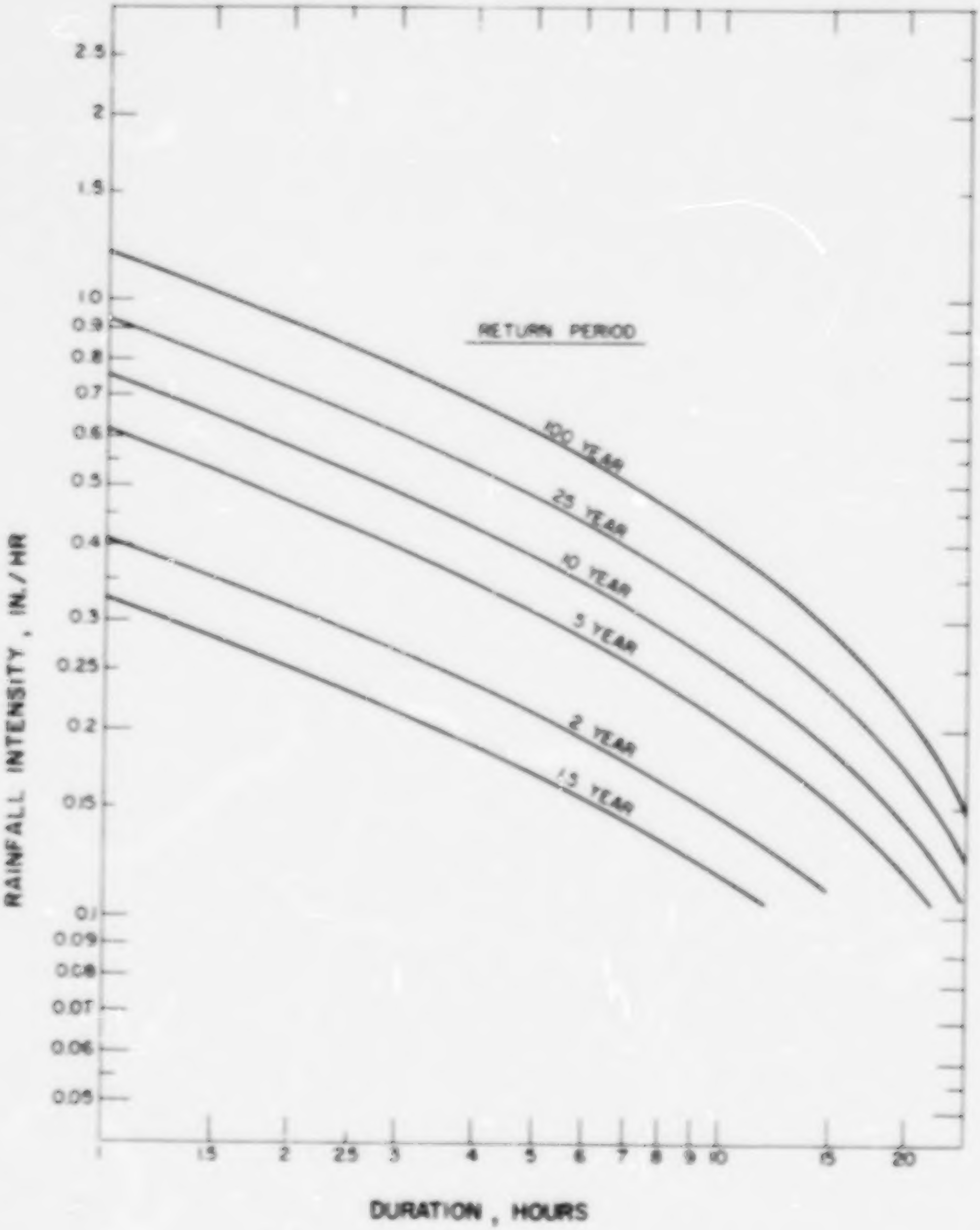

FIG. 20 INTENSITY - DURATION - FREQUENCY CURVES, OAKLAND AIRPORT GAGE, 1948-1968

SOURCE: BASIC DATA FROM E.S.S.A., HOURLY PRECIPITATION, CALIFORNIA 1948-1968

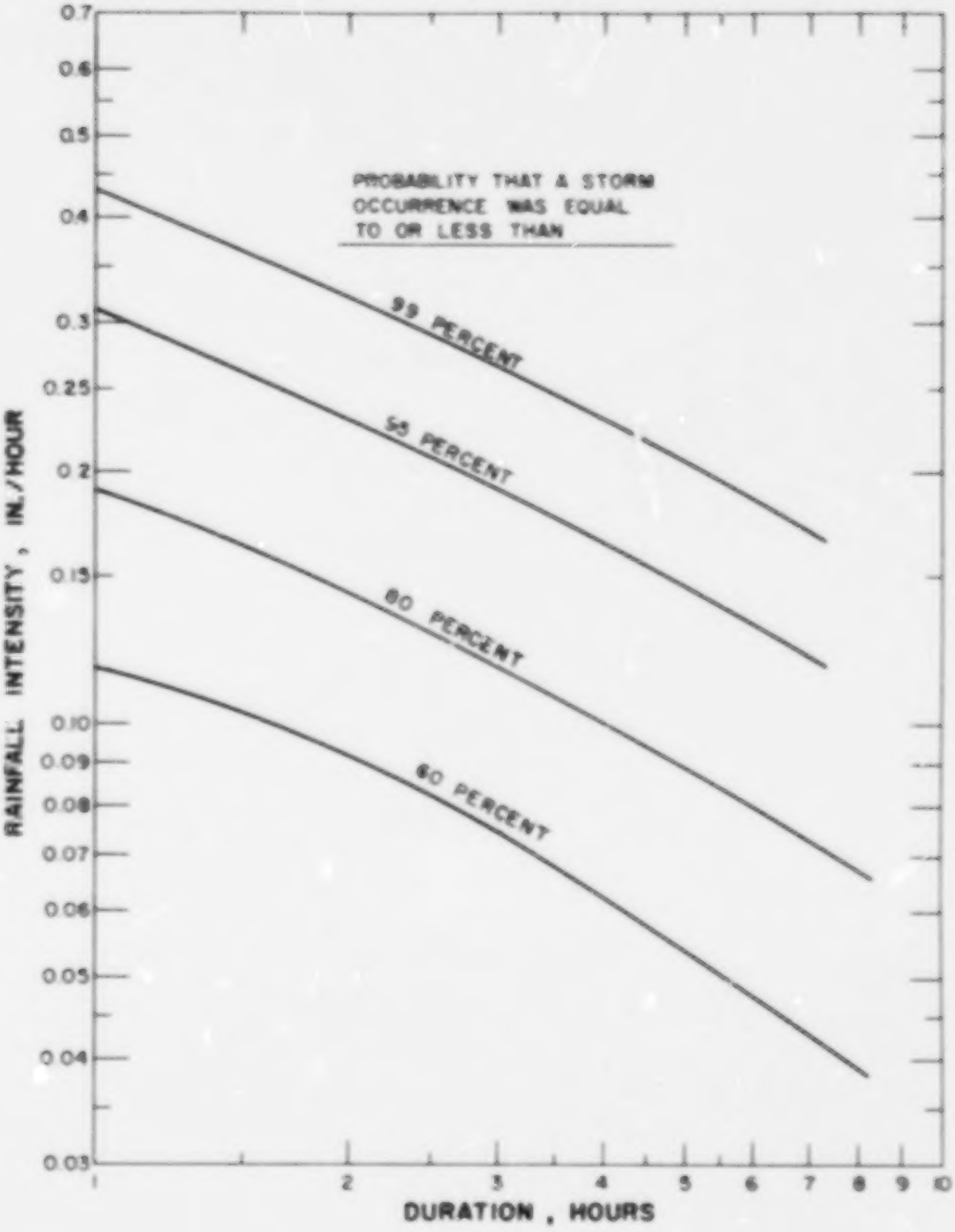

FIG. 21 INTENSITY - DURATION - PROBABILITY CURVES, OAKLAND AIRPORT GAGE, 1968-69 SEASON

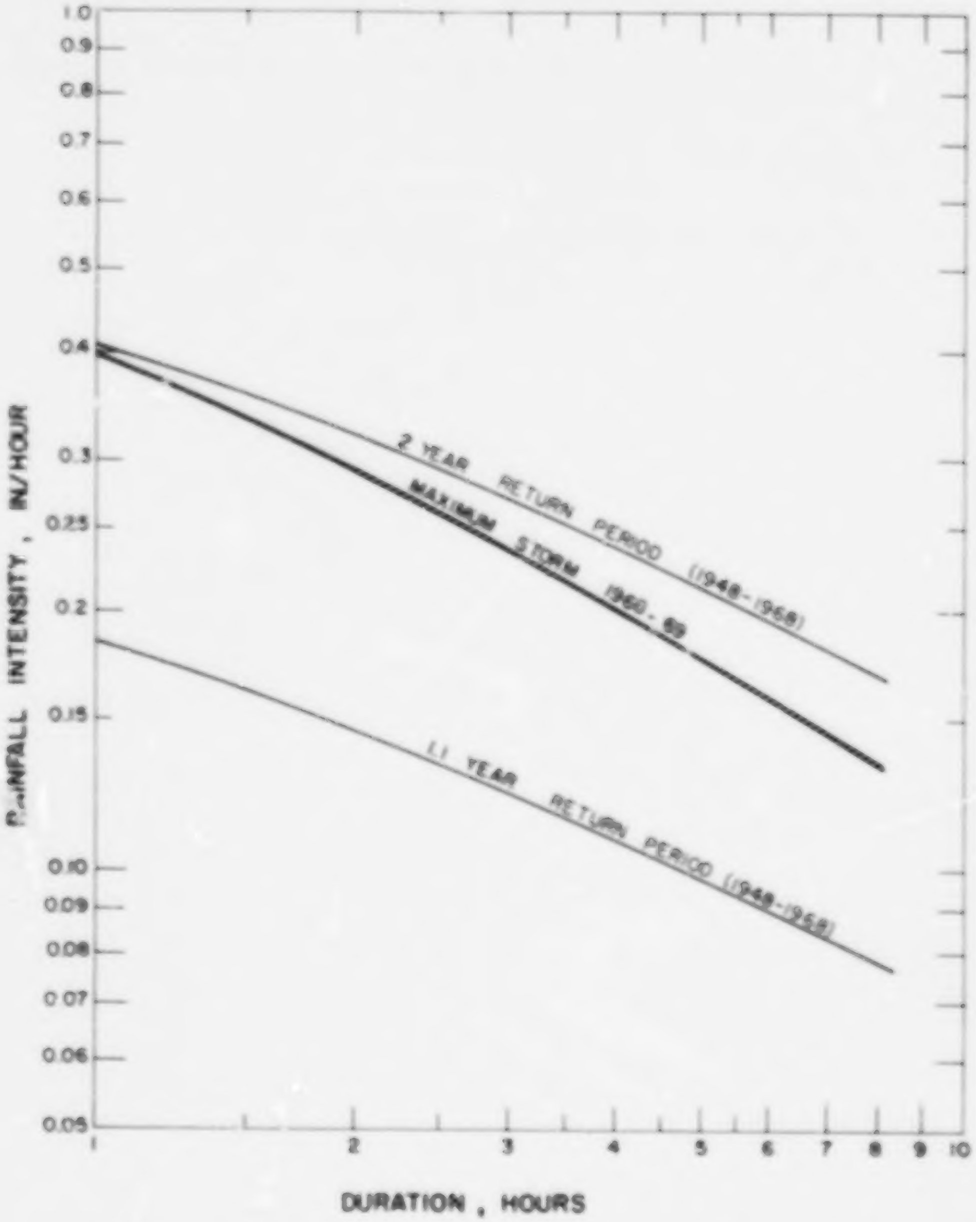

FIG. 22 FREQUENCY OF 1968-69 MAXIMUM STORM EVENT, OAKLAND AIRPORT GAGE

SOURCE: BASIC DATA FROM E.S.S.A., HOURLY PRECIPITATION, CALIFORNIA 1948-1968

DISTRIBUTION ANALYSIS

Two methods were used to describe the areal rainfall distribution over the study area:

1. An evaluation of total rainfall values for discrete storms during the 1968-69 season.

2. An evaluation of the mean annual precipitation values over the study area.

In addition to areal distribution, an attempt was made to describe the temporal distribution of rainfall over the study area. The methods and results are described in the following discussion.

In the first method, total rainfall values from each storm during the 1968-69 rainy season measured at each gage were plotted against the total rainfall values for the same storms measured at the Oakland Airport gage. A line of best fit was drawn through the points, realizing that some of the points represented atypical storms. These atypical points represented rain that fell predominantly in the north end of the study area (not in the south end). Less importance was placed on these points because such atypical storms do not cause the worst total infiltration conditions. The best fit for all of the points for each gage was a straight line passing through the origin. The graphs are shown on Appendix Figure II-4.

Since the line of best fit was a straight line, the factor needed to adjust the storm total at the Oakland Airport gage to that at any gage is a constant value. This factor was plotted on a map showing the location of all of the gages. Lines were then drawn representing the equal values of this distribution factor. These values were adjusted somewhat on the basis of topography and mean annual precipitation. This procedure minimized the possible gage errors. The plot of the areal rainfall distribution factors on a map of the study area is shown in Figure 23.

The second method was based on mean annual precipitation values, which were derived from an isohyetal map presently used by the Alameda County Flood Control and Water Conservation District. This map was based on 30 years of records between 1930-31 to 1959-60. Factors were computed for each gage location as accurately as the data could be read from the map. Table 13 presents a tabulation of the factors developed by the two methods. Some of the differences may be due to the following three points:

1. The comparison was made between discrete storm data versus mean annual data.

2. The comparison was made between measured and estimated data.

FIG. 23 AREAL RAINFALL DISTRIBUTION FACTORS

TABLE 12

AREAL RAINFALL DISTRIBUTION FACTORS

	Hourly gages*			Daily gages	
Locations	Factors from 1968-69 storms	Factors from mean annual rainfall	Locations	Factors from 1968-69 storms	Factors from mean annual rainfall
Alameda Naval Station	0.92	1.02	Chabot Filters	1.20	1.25
Berkeley (ESSA)	1.24	1.25	Chabot Reservoir	1.20	1.25
Capistrano Drive	1.10	1.15	Upper San Leandro Reservoir	1.33	1.40
Carleton Avenue	1.05	1.17	California School for the Blind	1.25	1.24
EBMUD Plant	0.90	0.87	Shepherd Canyon	1.45	1.26
Oakland International Airport Meteorological Station**	1.00	1.00	Skyline Boulevard	1.35	1.37
Oakland City Hall	1.01	0.94	Oakland, Glenwood Glade	1.35	1.26
Orinda Filters	1.55	1.55	Oakland, Shirley Drive	2.19	1.41
Piedmont	1.20	1.26	Oakland, McMillan	1.19	1.17
Pump Station 2	1.00	1.00	Oakland, 39th Avenue	1.23	1.23
Spruce Street	1.31	1.31			
Upper San Leandro Filters	1.22	1.32			

*Recording type gages.

**Base gage.

3. A more complete rainfall data collection network was available for the 1968-69 data than for establishing the map of the mean annual precipitation values.

The temporal rainfall distribution in the study area was analyzed by comparing the time at which the storm event began at each gage location. The results indicated that the storms generally began during the same hour over the study area. This conclusion is based on those storms that were general in nature and not on those that covered only certain portions of the study area. Because the rain gages recorded the rainfall variations on graphs with scales that could not be read at intervals much smaller than one hour, a better definition of temporal distribution was not possible.

DEFINITION OF STORMS CAUSING OVERFLOWS AND BYPASSES

Attempts to describe a "design storm" in the classical sense were fruitless. The term design storm involves consideration of intensity, duration, and frequency of rainfall; and the selection of frequency involves an understanding of the economic or destructive effects encountered when the event is exceeded in magnitude. The knowledge and understanding of the local citywide and districtwide effects that may result from overflows were felt to be inadequate for defining the design storm parameters. Also, no single design storm could be selected that would account for all of the various problems resulting from rainfall throughout the study area. An attempt was made, however, to describe the events that took place during the 1968-69 season by the storms which caused them. The main events for which records were available were the pump station overflows and the water pollution control plant bypasses.

The rainfall data taken at the Oakland Airport between November 1968 and April 1969 indicated that a total of 60 storm events took place. As mentioned previously, a storm event is defined as a block of rainfall equal to or greater than 0.02 inch, separated in time from any other event by at least five hours.

The first analysis involved an evaluation of the distribution of storm peaks throughout the day. Table 14 shows the number of storms at the Oakland Airport that had peaks during each of the six 4-hour periods of the day. The storms investigated represented 28 of the largest storms during the season. On the basis of the distribution of storm peaks indicated in Table 14, it was concluded that there was no need to define the storm by the period of day during which it occurred. This conclusion was significant because if the peak of a majority of the storms had occurred during the early morning hours (12 midnight to 7 a.m.) when the dry weather or base flow is lowest, the effect would have been minimized. No other attempts were made to relate storm peaks or characteristics to various periods of the day.

TABLE 14

OCCURRENCE OF RAINSTORM PEAKS DURING THE DAY, OAKLAND AIRPORT, 1968-69

4-Hour interval during 24-hour day	Number of peaks during the interval
0:01-4:00	8
4:01-8:00	3
8:01-12:00	7
12:01-16:00	4
16:01-20:00	2
20:01-24:00	4

The next step was to relate the magnitude of the storms to overflows recorded at Pump Stations A and B. The rainfall was plotted versus storm duration for the first four hours of the largest storms during the season. Storms that resulted in overflows at Pump Stations A and B, as reported by the operating staff, were labeled on the points of the graph. From these data it was possible to define three zones: an upper zone in which pump station overflows were certain, a middle zone in which overflows were questionable, and a lower zone in which overflowing would not occur. From the distribution of points in the middle zone, it was possible to draw in a line which represented storm conditions that could be on the borderline between causing overflows and not causing overflows at the pump stations. This borderline storm was termed the "marginal overflow storm" and is shown on Figure 24.

A similar analysis was made for describing storm conditions causing water pollution control plant bypassing. The results are shown in Figure 25. In this case the borderline storm was termed the "marginal plant bypass storm."

A close observation of Figures 24 and 25 will indicate that the middle zones for both plots are in essentially the same place and that the lines representing the two marginal storms are quite similar. For simplicity, the two marginal storms will be considered identical in subsequent analyses. This similarity in marginal storms suggested that the frequency of occurrence of overflows and bypasses is nearly the same.

Although not completely applicable, an attempt was made to relate the marginal overflow storm to the frequency or return period. Considering the marginal storm described in Figure 24 as a maximum event for a year, and referring to the frequency analysis shown on Figure 20, a return period of 1.08 years was selected. Therefore, the marginal overflow

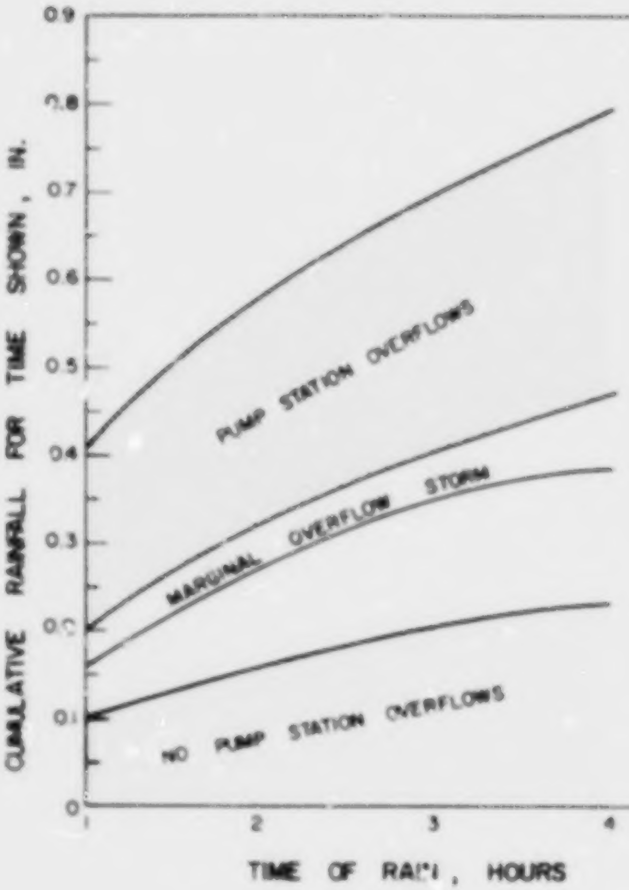

FIG. 24 MARGINAL OVERFLOW STORM, 1968-69

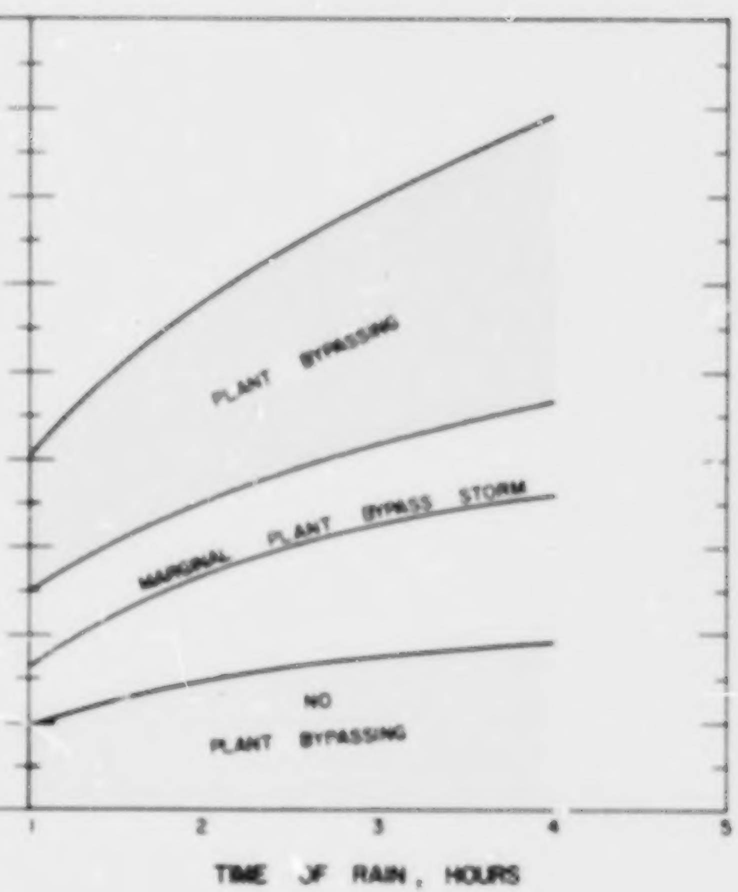

FIG. 25 MARGINAL BYPASS STORM, 1968-69

storm may be expected to be equalled or exceeded at least once every 1.08 years, i.e., equalled or exceeded 92 out of 100 years. The analogy is not completely proper because the marginal storm in the last season was not a maximum event. In fact, the marginal storm was exceeded by 16 other storms. Nevertheless, the analogy offers a basis for visualizing the characteristics of the marginal overflow storms.

SECTION 6

ESTIMATE OF THE VOLUMES OF STORM WATER INFILTRATION

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SECTION 6

ESTIMATE OF THE VOLUMES OF STORM WATER INFILTRATION

In the first part of this section, 1968-69 data were used to develop and check two methods--the extrapolation method and the land use method--for estimating extraneous flow from a storm of given characteristics.

The second part of this section deals with the estimation of total annual quantities of extraneous flows and the disposition of the various component volumes for the entire study area. The estimations were made by the land use method and the application of various historical flow data. The quantities were estimated for the 1968-69 season and for a year having average rainfall.

The significant conclusions developed in this section are outlined below and graphically summarized on Figures 26 and 27.

1. Relative magnitude of the sources of storm water infiltration:

   a. The equivalent storm water infiltration ratio for the East Bay Area is 0.111, i.e., 11.1 percent of the total volume of rain falling on the area enters the sanitary sewer system.

   b. Approximately 30.6 percent of the total volume of infiltration is contributed by infiltration and runoff from those areas which have combined sewers, composing 4 percent of the study area.

   c. Approximately 3.7 percent of the total volume of infiltration is contributed by the Pump Station A drainage area, composing 1.4 percent of the study area.

   d. In the remaining 94.6 percent of the study area, approximately 26.2 percent of the total infiltration is contributed via direct connections, and 39.5 percent is contributed via percolation or pipe leakage.

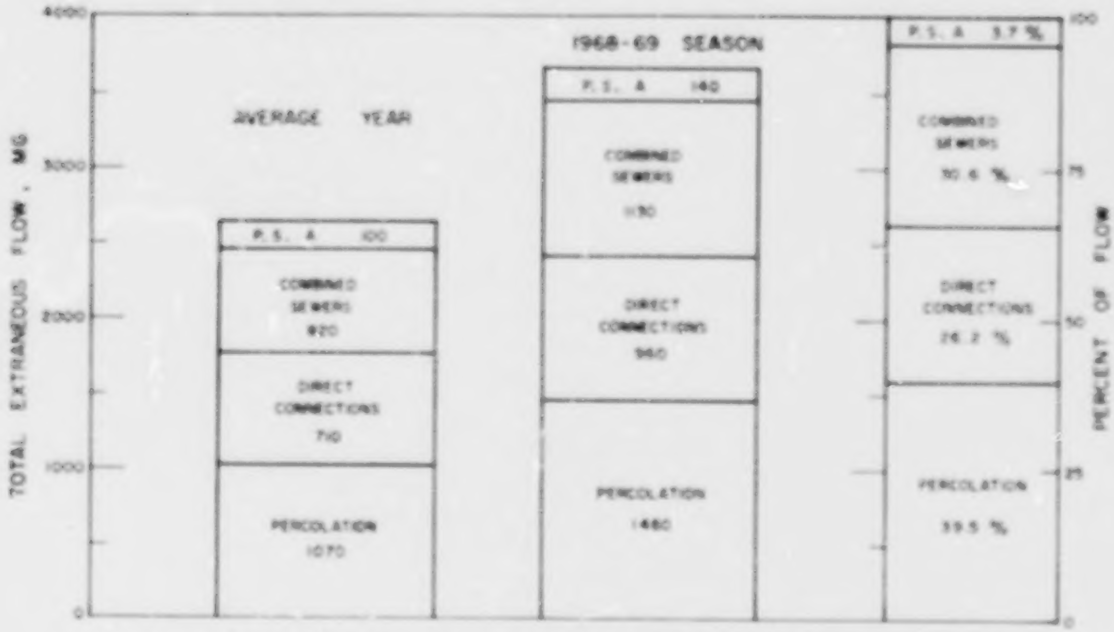

FIG. 26 SOURCES OF EXTRANEOUS FLOW, 1968-69 SEASON VERSUS AVERAGE YEAR

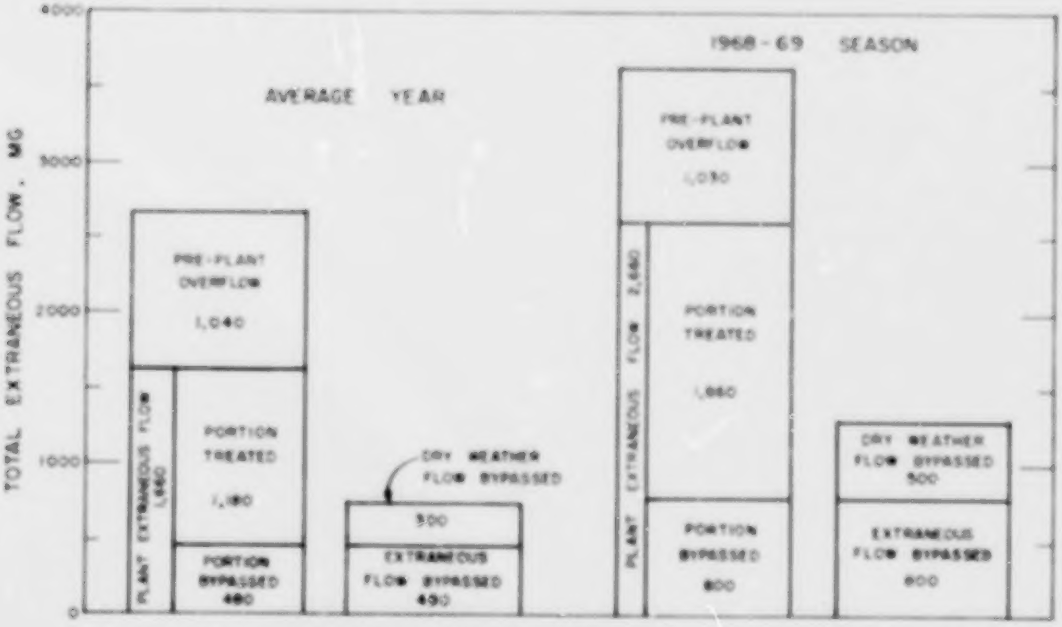

FIG. 27 DISPOSITION OF EXTRANEOUS FLOW, 1968-69 SEASON VERSUS AVERAGE YEAR

SOURCE: EBMUD WATER POLLUTION CONTROL PLANT ANNUAL REPORTS AND LABORATORY DATA

2. Overflows and bypasses during an average year:

   a. The total rainfall is 17.57 inches at the Oakland Airport.

   b. Bypasses were found to occur 11 or 12 times, and it was concluded that overflows would occur the same number of times.

   c. Approximately 1,660 MG of extraneous flow (6.8 percent of the rainfall) reach the water pollution control plant and 1,040 MG of extraneous flow (4.3 percent of the rainfall) overflow before reaching the plant.

   d. Bypassing at the plant occurs for 110 hours at an average rate of 168 mgd, resulting in a discharge of 780 MG of waste water that consists of 480 MG of extraneous flow and 300 MG of base or dry weather flow.

   e. Approximately 1,180 MG (1,660 - 480) of extraneous flow reaching the plant are treated before discharging.

   f. Overflow volumes were estimated to equal 90.5 MG per event, with 62.5 MG occurring south of the plant and 28.0 MG occurring north of the plant.

3. Overflows and bypasses during the 1968-69 season:

   a. The total rainfall was 24.1 inches at the Oakland Airport.

   b. Approximately 2,660 MG of extraneous flow (8.0 percent of the rainfall) reached the water pollution control plant, and 1,030 MG (3.1 percent of the rainfall) overflowed before reaching the plant.

   c. Bypassing occurred for 186 hours at an average rate of 168 mgd, resulting in a discharge of 1,300 MG of waste water that consisted of 800 MG of extraneous flow and 500 MG of base or dry weather flow.

   d. Overflow volumes at Pump Stations A and H accounted for 3.5 percent of the total overflow volumes. The remaining 96.5 percent was assumed to occur at various overflow structures along the interceptor and within the sewer system.

DEVELOPMENT OF METHODS FOR ESTIMATING VOLUMES OF OVERFLOWS

As mentioned, two methods were used to estimate storm water infiltration quantities in the study area for a specific storm event that occurred during the 1968-69 season: 1) the extrapolation method and 2) the land use method. The extrapolation method involved the use of the flow routing program for computing the outflow hydrograph and the quantity of sewer system overflows in a pilot drainage area. The results from this pilot drainage area were then extrapolated to a larger drainage area, of which it was a part, and finally to the entire study area. The land use method involved the application of gross infiltration ratios to the rainfall volumes over each land use category in the study area. Results from both methods were compared and checked by measured quantities. The extrapolation method was used to estimate the relative magnitude of the sources of infiltration. The land use method could only be used to estimate the total volumes of infiltration, without regard to source.

THE EXTRAPOLATION METHOD

Use and Confirmation of the Method

The first step in the extrapolation method was to select and describe the pilot drainage area. Drainage Area No. 29, located in the south part of Oakland, was chosen because it is part of the total drainage area (composed of Nos. 29, 30, 31, 32, 33, and 34) that contributes to Pump Station H, where extensive flow measurements were made during this investigation. These flow data were thus available for checking the results. Drainage Area No. 29 is shown in Figure 28. It contains the seven combinations of land use, topography, and age of sewer for which infiltration ratios were developed in Section 5. Since this area stretches from the hillside across Oakland to the waterfront, it also contains most of the topographic conditions found in the East Bay Area. The areal rainfall distribution factors for this drainage area vary from 1.3 at the upper end to 1.0 at the lower end, as can be seen on Figure 23 in Section 5.

Once the drainage area was chosen and its boundaries determined, the most difficult task was to describe its pipe network, which is probably similar to that in many older parts of the East Bay Area. As-built sewer drawings were obtained from the City of Oakland, but these were found to be incomplete. One of the major problems in describing the pipe network was the existence of parallel sewers (new sewers laid parallel to older sewers to increase the capacity). In some instances, data regarding the interconnections between these pipes were missing on the drawings and therefore additional investigations were required. In other instances, data on invert elevations, on sewer sizes, and even on pipe locations were also missing.

FIG. 28 LAND USE DISTRIBUTION, DRAINAGE AREA NO. 29

SOURCE: OAKLAND CITY PLANNING COMMISSION

The final sewer network used in the computer program, as shown on Figure 29, was made up of 177 pipe lengths, each representing an average of 5.5 acres of drainage area. Parallel pipes were reduced to hydraulically equivalent single pipes. The drainage area for each pipe was further described by the amount of land area involved in each of the seven types of land use categories and by the areal rainfall distribution factor.

The second step in the extrapolation method was to select the storm characteristics for which flow measurements were taken both at Pump Station H and at the water pollution control plant. Only two discrete storms having conditions suitable for computation and extrapolation were found. All other storms were excluded 1) because of interference by either previous or subsequent storms, or 2) because flows were not measured at both Pump Station H and the main plant. Of the two suitable storms, the one selected for use in the typical computations was for February 4-5, 1969. This storm, which had 0.48 inch of rain, was exceeded in total volume 30 percent of the time during the 1968-69 season, but was exceeded in average intensity only 10 percent of the time.

The third step in the extrapolation method was to run the computer program for the pilot drainage area. The printout from the program included the depth of flow in each sewer reach for the peak flow rate, together with the velocity and the sewer capacity. The listing of these data permitted a quick review of the oversized sewers as well as any bottlenecks in the system. The output indicated that 8 of the 177 pipes were surcharged and perhaps overflowing. The question of surcharging could not be better defined because of the method, described below, for computing overflows. Many of the pipes were found to be flowing less than half full even at the peak flow rate, indicating that storage capacity might be available, depending upon the ground elevations. These pipes were in random locations, and the beneficial use of their natural storage capacity would not be easily effected.

The computer program was written with due consideration for the evaluation of overflows within the sewer system. If the peak flow rate of the computed hydrograph would not pass through the next downstream sewer without surcharging, the hydrograph would be recomputed by making use of more of the available storage volume in the first upstream reach of each intersecting sewer. The value stored during each routing period is released to the downstream sewer during the next routing period, thus changing the hydrograph. If the upstream pipes were full and the resulting dampened hydrograph still could not pass through the downstream sewer, the program would compute the excess amount of flow as an overflow, and the remainder of the hydrograph would be routed on through the system. Overflow quantities thus obtained may not actually occur, however, because nominal surcharging can increase the hydraulic capacity of the downstream section so that all of the water may pass on through without overflowing. In spite of the limiting assumptions and approximations necessary for this method

FIG. 29 SANITARY SEWER NETWORK, DRAINAGE AREA NO. 29

SOURCE: OAKLAND SEWER MAINTENANCE DEPARTMENT

of overflow computation, the computed volumes seemed to compare well with the measured volumes when extrapolated to the total drainage area of Pump Station H.

The computed volume of extraneous flow for Drainage Area No. 29 was 1.23 MG, with 0.98 MG reaching the end of the sewer and 0.25 MG overflowing within the system. The output from the program could not be directly checked against measured data because the discharge point was not one of the measuring stations. Therefore, in order to check the results, the computed volume of water was extrapolated to the total Pump Station H drainage area, of which Drainage Area No. 29 is a part.

The final step was to extrapolate the extraneous flow volume computed for Drainage Area No. 29 to the Pump Station H drainage area. The computed volume was multiplied by the ratio of the land use product for the Pump Station H drainage area to the land use product for Drainage Area No. 29. The development of the land use products is shown in Table 15. The land use products were computed by multiplying the infiltration ratio by the land area over which the infiltration ratio is applicable, and by the areal rainfall distribution factor. The overall land use product was found by summing the individual products for each land use. (The infiltration ratio for each land use category is given in Table 12). Consequently, the extrapolation factor for Drainage Area No. 29 to the Pump Station H drainage area was 10.5 (1,136/108 = 10.5). Multiplying the computed volume of extraneous flow from Drainage Area No. 29 by the ratio of 10.5, gave the volume of extraneous flow estimated for the Pump Station H drainage area during the February 4-5 storm as 10.2 MG. The measured volume[4] of extraneous flow at Pump Station H during that storm was 10.1 MG, which compares reasonably well with the computed value.

Multiplying the computed volume of sewer system overflows, 0.25 MG, by the ratio of 10.5, gave the volume of water derived from storm water infiltration that never reached Pump Station H as 2.60 MG. Therefore, the total computed volume of storm water infiltration for the drainage area tributary to Pump Station H was 12.8 MG (10.2 + 2.6).

A similar procedure was then used to estimate the extraneous flow for the total study area from that same storm. The infiltration ratios, land use areas, and areal rainfall distribution factors used in this analysis are listed in Table 16, together with the computational steps needed to develop the total adjusted land use product. The ratio of the land use product for the total study area as compared to the land use product for Drainage Area No. 29 was found to be 52.5. Assuming that the proportion of sewer system overflows is the same for Drainage Area No. 29 as for the total study area and that no other overflows are in operation, the total extraneous flow reaching the water pollu-

[4]Obtained by subtracting average base flow from measured wet weather flow.

TABLE 16

LAND USE PRODUCTS FOR PUMP STATION H AREA AND DRAINAGE AREA NO. 29

Area description	Gross infiltration ratio (1)	Land area, acres (2)	Areal rainfall distribution factor (3)	Product (1)x(2)x(3), acres (4)
Pump Station H Area				
Residential				
Low density				
Old*, hilly	0.045	4,490	1.26	254
New, hilly	0.015	400	1.40	8
Old, flat	0.110	420	1.17	54
Medium density	0.120	3,790	1.15	523
Commercial	0.090	1,050	1.15	109
Industrial	0.085	2,110	1.05	188
Totals				1,136
Drainage Area No. 29				
Residential				
Low density				
Old, hilly	0.045	200	1.26	11
New, hilly	0.015	--	--	--
Old, flat	0.110	60	1.15	8
Medium density	0.120	480	1.15	66
Commercial	0.090	210	1.07	21
Industrial	0.085	20	1.05	2
Totals		970		108

*Refers to age of sewer system.

TABLE 16

LAND USE PRODUCTS FOR STUDY AREA WITH AND WITHOUT COMBINED SEWER AREAS

Area description	Gross infiltration ratio (1)	Land area, acres (2)	Areal rainfall distribution factor (3)	Product (1)x(2)x(3), acres (4)
A. Study area less combined sewer areas				
Residential				
Low density				
Old, hilly	0.045	13,100	1.30	766
New, hilly	0.015	400	1.40	8
Old, flat	0.110	3,040	1.05	351
Medium density	0.120	8,960	1.13	1,215
Commercial				
Oakland downtown	0.140	1,700	1.10	262
Elsewhere	0.090	5,530	1.08	536
Industrial	0.085	8,870	1.03	778
Pump Station A	0.246	710	1.20	209
Subtotals		42,310		4,125
Averages	0.0812*		1.20	
B. Total study area				
Combined sewer areas	0.700	2,060**	1.20	1,730
Remaining area	0.0812	40,250	1.20	3,920
Equivalent values	0.111***	42,310	1.20	5,650

*Computed value: 0.0812 = 4,125/1.2x42,310.

**Four percent of the total area: 2,060 = 0.04x51,416 acres.

***Computed value: 0.111 = 5,650/1.2x42,310.

tion control plant would be 51.1 MG and the sewer system overflows would be 13.0 MG. The volume of extraneous flow measured at the plant[5] during that storm was 50.5 MG. The close agreement of the computed extraneous flow and the amount observed for this storm supports the validity of the extrapolation method.

Relative Magnitude and Sources of Infiltration

In developing the land use product for the total study area, special consideration was given to the Pump Station A drainage area and to the portions of the study area that still have combined sewers. The method used in accounting for these areas is shown in Table 16. These two areas were given special attention because their infiltration was known to be greater than would be indicated by use of the infiltration ratios developed in the study subareas. Flow data from Pump Station A gathered during the 1968-69 season indicated a gross infiltration ratio of 0.246, which is substantially greater than any other ratio measured during the season. The estimate that 4 percent of the study area has combined sewers was based on reports from personnel of EBMUD and the various cities. These combined sewer areas were assumed to have an "infiltration ratio" of 0.70 (i.e., equivalent to a storm water runoff coefficient).

Considering the additional infiltration contributed from both the Pump Station A drainage area and the combined sewer areas, the equivalent infiltration ratio for the total study area was found to be 0.111, as shown on Table 16. This figure is significant because it indicates that 11.1 percent of the total volume of rain falling on the study area enters the sanitary sewer system, the only exceptions being those portions that do not contribute to sanitary sewers.

It is important to note from the land use products in Table 16 that the combined sewer areas, consisting of only 4 percent of the study area, account for about 30.6 percent of the total amount of water defined as infiltration (1,730 x 100/5,650 = 30.6%).

It is also important to note that the Pump Station A drainage area, consisting of only 1.4 percent of the study area, accounts for 3.7 percent of the total amount of infiltration (210 x 100/5,650 = 3.7%). Therefore, these areas, the combined sewers and Pump Station A, should be given priority when considering ways of reducing storm water infiltration in the East Bay Area.

Table 17 develops the land use products using only the direct connection infiltration ratios in a manner similar to that using the gross infiltration ratios in Table 16. The adjusted land use product in Table 17 excludes the Pump Station A drainage area and the combined sewer areas. Comparing the direct connection land use product to the total land use product, it will be seen that direct connections con-

[5] Obtained by subtracting average base flow from measured wet weather flow.

TABLE 17

LAND USE PRODUCTS USING DIRECT CONNECTION INFILTRATION RATIOS

Area description	Direct connections infiltration ratio* (1)	Land area, acres (2)	Areal rainfall distribution factor (3)	Product (1)x(2)x(3), acres (4)
Residential				
Low density				
Old, hilly	0.02	13,100	1.30	342
New, hilly	0.01	400	1.40	6
Old, flat	0.02	3,040	1.05	64
Medium density	0.02	8,960	1.13	202
Commercial				
Oakland downtown	0.10	1,700	1.10	187
Elsewhere	0.05	5,530	1.08	298
Industrial	0.05	8,870	1.03	458
Totals		41,600		1,557
Combined sewers		2,060*		
Adjusted product		39,540		1,480**

Percent of total infiltration volume contributed by direct connections $= \frac{\text{This product}}{\text{Total product}} = \frac{1,480 \times 100}{5,650} = 26.2\%$

Percent of total infiltration contributed by percolation $= \frac{(3,920-209)^{***} - 1,480}{5,650} \times 100 = 39.5\%$

*See Table 12.

**1,480 = (39,540/41,600) x 1,557.

***See Table 16.

tribute approximately 26.2 percent of the total infiltration volume. Table 17 also shows that the percolation source of infiltration over the total study area, excluding the Pump Station A drainage area and the combined sewer areas, amounts to 39.5 percent of the total infiltration volume. This indicates that the percolation source of infiltration contributes 51 percent more water than the direct connection source of infiltration ((39.5-26.2)100/26.2 = 51%).

Table 18 shows the relative proportions of infiltration quantities expected north and south of the water pollution control plant. It was assumed that the ratio of the land use products for the areas north and south of the water pollution control plant are indicative of the relative quantities of infiltration and also of overflows north and south of the plant. As shown in Table 18, 31 percent of the extraneous flow will occur north of the plant and 69 percent will occur south of the plant.

THE LAND USE METHOD

The land use product, as used previously, is expressed in units of acres. Because the land use product contains an adjustment for rainfall distribution and for infiltration ratios, the direct multiplication of rainfall quantity by land use product should equal the total volume of infiltration from any given storm. The storm used for the previous computations contained 0.48 inches of rain. Using the total land use product of 1,136 acres for the Pump Station H drainage area, multiplying by the rainfall of 0.48 inches and correcting the units, the estimated total volume of infiltration in the Pump Station H drainage area would be 14.8 MG (0.48 x 1,136 x 0.0271 = 14.8). The volume computed by the extrapolation method was 12.8 MG. The difference of 2.0 MG was partly caused by not considering the loss of the first 0.05 inch of rainfall for the percolation portion of the infiltration which was done in the flow routing program. Considering this loss, the difference between the two methods is reduced to 0.9 MG, which may be attributable to the mechanics of the routing program, the method of handling the surcharged condition, and the computation of resulting overflows.

A similar procedure was then used to estimate the extraneous flow for the total study area from the same storm. By using the equivalent land use product developed in Table 16, applying the rainfall of 0.48 inch and correcting the units, the total infiltration quantity was 73.5 MG (5,650 x 0.48 x 0.0271). Recomputing the infiltration volume leaving out the first 0.05 inch of rain that enters via percolation, the total expected infiltration by the land use method was 67.5 MG, which may be compared to the total of 64.1 MG found by the extrapolation method.

These calculations indicated that the land use method would provide results comparable to those from the extrapolation method, although somewhat higher. The land use method is quicker and less time consuming than the extrapolation method, but it provides a gross estimate

TABLE 18

LAND USE PRODUCTS FOR NORTH VERSUS SOUTH INTERCEPTOR AREAS

Area description	Gross infiltration ratio (1)	North: Land area, acres (2)	North: Areal rainfall distribution factor (3)	North: Product (1)x(2)x(3), acres (4)	South: Land area, acres (5)	South: Areal rainfall distribution factor (6)	South: Product (1)x(5)x(6), acres (7)
Residential							
Low density							
Old, hilly	0.045	4,760	1.30	278	8,340	1.30	487
New, hilly	0.015				400	1.40	8
Old, flat	0.110	700	1.17	90	2,340	1.01	260
Medium density	0.120	2,420	1.15	334	6,540	1.12	878
Commercial							
Oakland downtown	0.140				1,700	1.10	262
Elsewhere	0.090	2,290	1.10	227	3,230	1.07	312
Industrial	0.085	1,510	1.05	135	7,360	1.03	645
Pump Station A	0.246	710	1.20	209			
Totals				1,273			2,852
Percent of total				31%			69%

of total volumes, with no indication of the fraction of sewer system overflows. Had time been available during this study, the extrapolation method could have been used to estimate the actual shape of the infiltration hydrograph at all points along the interceptor, and thus provide a more complete picture of the events of overflowing and bypassing.

ESTIMATES OF VOLUMES OF OVERFLOWS AND BYPASSES

Estimates of storm water infiltration quantities for the average year were derived by the same land use method that was used to derive the volumes of discrete storms described in the foregoing discussion. This method, however, will not provide an indication of the disposition of the various portions of infiltration. The procedures used for defining the volumes and disposition of the total annual volume of infiltration are described in the discussion that follows.

The mean seasonal precipitation at the Metropolitan Oakland International Airport is 17.57 inches. Using the equivalent land use product developed in Table 16, which accounts for land area, gross infiltration, and areal rainfall distribution, the average annual volume of storm water infiltration in the study area was estimated to be 2,700 MG (17.57 x 5,650 x 0.0271). This estimated volume was based on the assumption that the average infiltration ratio for the study area is 0.111, which is actually applicable to only those conditions that prevailed during the 1968-69 season. The value will be expected to change as combined sewers are eliminated and as other improvements are made to reduce infiltration in the sewer system. Nevertheless, this value of infiltration ratio will be assumed applicable for the next several years.

ANNUAL EXTRANEOUS PLANT FLOW

Data were gathered and analyzed to determine the amount of extraneous flow which actually reaches the water pollution control plant as a result of storm water infiltration. The plant records of total monthly flow for the period from 1959 to 1969 were analyzed. Seasonal contributions of waste water from local canneries were found to add substantially to the average dry weather flows. Lines of best fit were drawn through plotted values of dry weather flow for both the canning and non-canning season. The dry weather flow months were selected from those that had an insigificant amount of rainfall, usually April, May, June, and July. The canning season months were assumed to be August, September, and October.

These lines of best fit were straight and parallel, as shown in the upper curve of Figure 30. The uniform slope of the line indicates a constant annual increase in dry weather flow. No annual increase in cannery waste water flow was indicated because the lines were parallel. A second graph was constructed

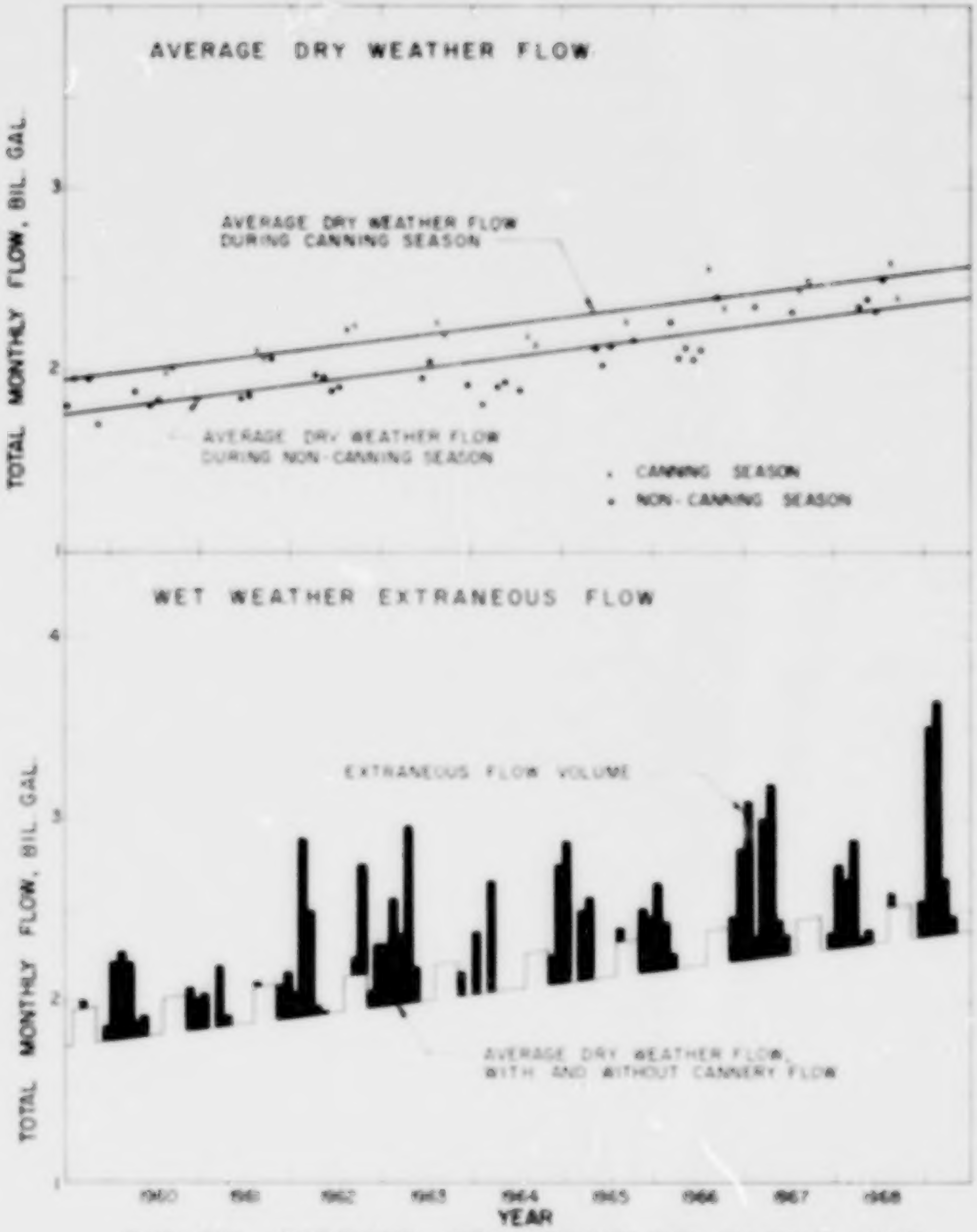

FIG. 30 AVERAGE DRY WEATHER FLOWS AND EXTRANEOUS FLOWS VERSUS YEAR, EBMUD WATER POLLUTION CONTROL PLANT

SOURCE: EBMUD WATER POLLUTION CONTROL PLANT ANNUAL REPORTS, 1959-1968

from the data for all months that had significant rainfall, as shown in the lower portion of Figure 30. That portion of the monthly flow falling above the dry weather flow conditions determined above was assumed to represent extraneous flow caused by the infiltration of storm water.

The flow data recorded during 1968-69 at the water pollution control plant, compared to the results of a more detailed analysis, appeared to be high. The actual values of flow were estimated to be approximately 85 percent of those recorded from pump running time computations (see Section 4 for discussion). It was assumed that this factor of 85 percent was applicable to previous years as well.

Therefore, the values for extraneous plant flow during 1959 through 1969, as derived from the lower curve of Figure 30, were adjusted by the factor of 85 percent. The resulting values were plotted versus annual rainfall measured at the Oakland Airport, and are shown in the lower curve of Figure 31. The curve drawn through these points indicates that, as expected, the ratio of extraneous flow to total annual rainfall tends to decrease during years of very high rainfall. This relationship is noted by the flattening out of the curve. The reason for this is that during periods of high rainfall, much more water is overflowed via system overflows, hence never reaches the plant.

Referring again to the lower curve of Figure 31, the average annual extraneous plant flow was found to be 1,660 MG for a mean seasonal precipitation of 17.57 inches measured at the Oakland Airport. From the same curve, the amount of extraneous plant flow during the 1968-69 season was found to be 2,660 MG, based on a total precipitation of 24.1 inches at the Oakland Airport.

ANNUAL PLANT BYPASSING

From the water pollution control plant records, it was found that the plant was bypassed a total of 186 hours during the 1968-69 season, amounting to about 1,300 MG, of which 500 MG was the base or dry weather flow (both the extraneous flow and the base or dry weather flow are bypassed). Therefore, the amount of extraneous flow bypassed was 800 MG. This amount of excess flow bypass results in an average flow rate during bypassing of 168 mgd.

Again, using the plant data for the 1959-69 period, a relationship between hours of plant bypassing and annual rainfall was developed, as shown in the upper curve of Figure 31. It is noted that mean annual precipitation is normally recorded for calendar years, whereas the reference in Figure 31 is to mean annual precipitation based on the fiscal year, July through June. The average annual duration of plant bypassing can be read from the curve as 110 hours based upon the annual precipitation of 17.57 inches. If the average bypass flow rate found for the 1968-69 season is assumed to represent average annual conditions, the average annual plant bypass is 780 MG (110 x 168/24). Again,

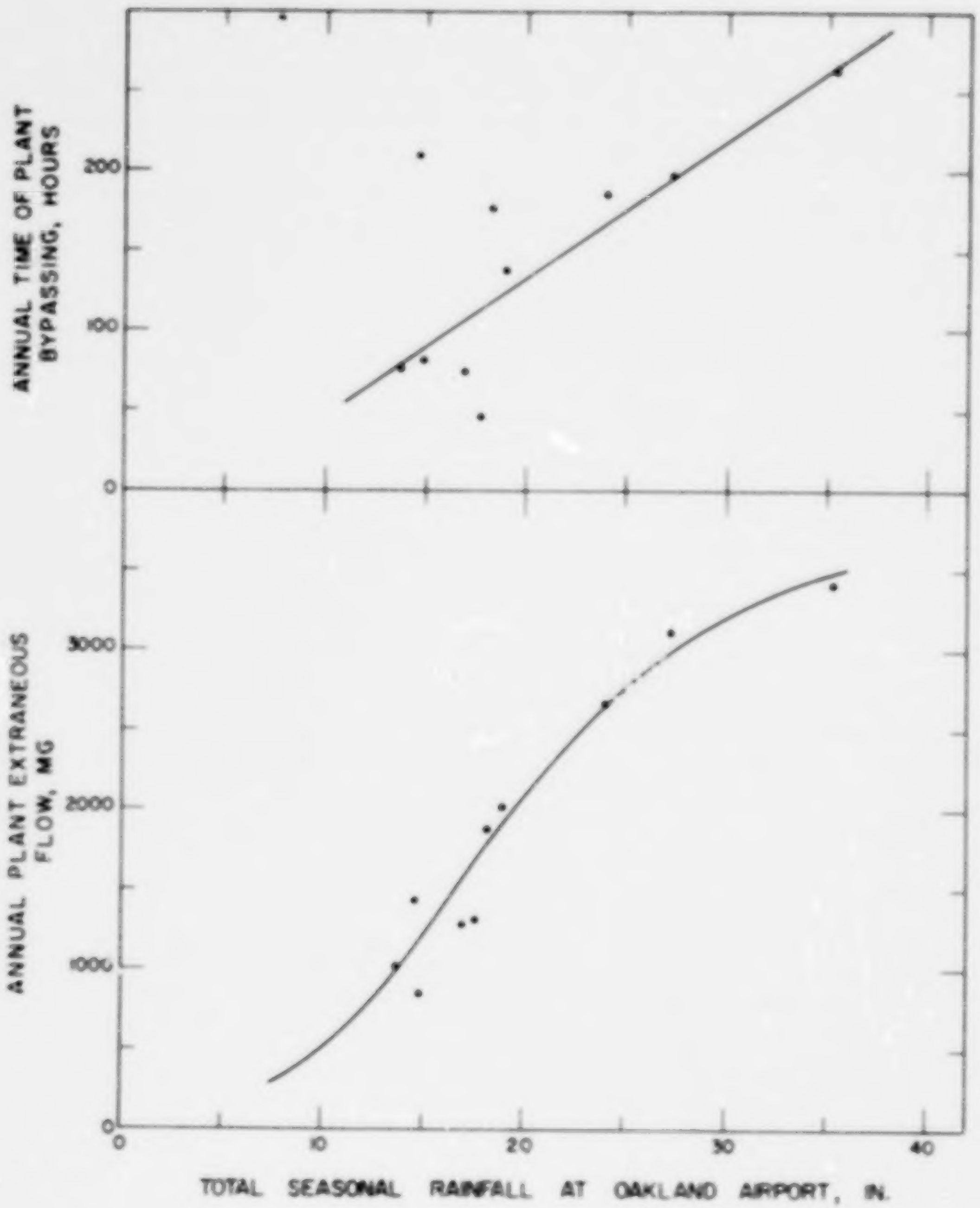

FIG. 31 ANNUAL PLANT EXTRANEOUS FLOW AND ANNUAL TIME OF PLANT BYPASSING VERSUS ANNUAL RAINFALL

SOURCE: BASIC DATA FROM EBMUD WATER POLLUTION CONTROL PLANT ANNUAL REPORTS, 1958-1969

assuming 1968-69 season conditions of 110 hours of plant bypassing at an average base flow rate of 66 mgd, approximately 300 MG of dry weather flow would be bypassed (110 x 66/24). The difference between the total plant bypass amount (780 MG) and the dry weather flow bypass amount (300 MG) of 480 MG represents the amount of extraneous flow which is bypassed annually.

The average number of yearly bypass occurrences can be determined by an analysis of the marginal overflow storms discussed in Section 5. The number of occurrences was found by counting the number of storms during the period from 1951-52 to 1968-69 in which the cumulative rainfall for each of the four hours of highest rainfall intensity was greater than the marginal overflow storm. The average number of occurrences was found to be 11.5 per year, i.e., 11 or 12 times per year.

The average annual volume of plant bypass per occurrence was found to be 68 MG (780/11.5 = 68). The assumptions used to determine this amount, however, do not take into account future changes. For example, it is known that the average annual dry weather flow in the future will not be equal to that in 1968-69. In fact, Figure 30 indicates that the future dry weather flow is likely to increase at a rate of about 1.8 mgd per year. An increase in the dry weather flow would decrease the amount of infiltration because of the reduced carrying capacity of the various sewers, but it would increase the amount of pre-plant overflowing, the number of times in which overflowing and bypassing take place and their duration, the volume of bypassing, and the amount of dry weather flow in the volume bypassed. The effect of the increasing dry weather flow on these values cannot be easily estimated. An attempt was made to determine the effect of increasing dry weather flow on the extraneous plant flow and the hours of bypassing for the water pollution control plant data from 1959 to 1969, but no relationship could be found.

ANNUAL OVERFLOWS

As previously discussed, the equivalent infiltration ratio computed in Table 16 representing total infiltration over the study area was 0.111. A similar computation was made indicating that the annual plant extraneous flow could be represented by an equivalent infiltration ratio of 0.08. The difference between these two infiltration ratios, 0.031, represents the volume of rainfall assumed to have overflowed prior to reaching the water pollution control plant. Therefore, the estimated quantity of storm water infiltration suspected of overflowing prior to reaching the plant was 1,030 MG (0.031 x 2,660/0.08 = 1,030) for the 1968-69 season. Similarly, the average annual overflow volume was estimated to be 1,040 MG per year, assuming that the 1968-69 relationships continue to exist. This value was found by subtracting the amount of extraneous plant flow expected annually (1,660 MG) from the total amount of infiltration found by application of the land use method (2,700 MG).

The average number of times per year that overflowing is expected to occur is the same as for bypasses, i.e., 11.5. This conclusion is based on the fact that the marginal overflow storms are nearly identical. It was assumed that the largest volume of overflow ahead of the plant would occur at the same time that pump stations overflow. Therefore, the number of all pre-plant overflows is assumed to be the same as that for pump station overflows. It is noted, however, that some pre-plant overflows will take place before pump station overflowing begins, and also at times when no pump station overflowing occurs. The average overflow amount per occurrence was estimated to be 90.5 MG (1,040/11.5). The amount of overflowing that occurs from the areas north and south of the water pollution control plant, respectively, is not actually known. It was assumed that the volumes of overflow would be proportional to the ratios of infiltration between the two areas. The ratios of infiltration were computed in Table 18 as 31 percent north and 69 percent south of the plant. Therefore, of the 90.5 MG of average annual overflow, 28.0 MG overflow in the area north of the plant, and 62.5 MG overflow in the area south of the plant.

Overflows at Pump Stations A and H

The flow data gathered for Pump Station A for the 1968-69 season indicated that 1.22 MG of overflow occurred between December 8 and February 17. During this period, 14.46 inches of rain fell at the Oakland Airport. If 24.6 percent of the rain infiltrated, a total volume of 82 MG of infiltrated flow would have resulted. Of this 82 MG, 1.22 MG were indicated as overflowing, which is 1.5 percent of the total infiltration. If this percentage of overflow were to hold for the rest of the season during which overflows were not measured, a total of 2.5 MG would be expected to overflow. This 2.5 MG of overflow represents 0.24 percent (2.5 x 100/1,030 = 0.24%) of the total estimated overflow for the study area during 1968-69.

The plant records indicate that 34 MG overflowed at Pump Station H during the past season. This volume represents 3.3 percent (34 x 100/1,030 = 3.3%) of the total amount of overflow for the study area during 1968-69.

All Other Overflows

The quantity of overflows during the 1968-69 season from other than the pump stations accounts for 994 MG. This figure represents 96.5 percent of the total amount of overflows during 1968-69.

SECTION 7

PROBLEMS RESULTING FROM OVERFLOWS

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SECTION 7

PROBLEMS RESULTING FROM OVERFLOWS

The problems that result from overflows in the East Bay Area may be categorized according to the type of overflow or bypass involved. Four types of overflows were found:

1. Intersystem overflows - occurring at cross-connections between storm drains and sanitary sewers.

2. System overflows - occurring at specially designed structures near the interceptor sewer to discharge excess flow to the Bay.

3. Water pollution control plant bypass - manually controlled bypass to prevent damage to the plant's mechanical equipment.

4. Intrasystem overflows - occurring at manholes or other openings and broken sewers.

Intersystem and system overflows and plant bypasses ultimately result in discharging untreated sewage to San Francisco Bay. The problems associated with discharges to the Bay will be considered in detail in Section 9.

Intrasystem overflows, which commonly occur throughout the East Bay cities, do cause serious problems (involving public health, community hygiene, property damage, and nuisance), but they do not have a significant direct pollutional effect on San Francisco Bay. Therefore, these problems are not considered in the evaluation of pollutional effects in Section 9 but alternatives for their correction are discussed in Section 10.

INTERSYSTEM OVERFLOWS

Several problems are associated with the intersystem type of overflow. Raw sewage is discharged into storm drains that lead directly to the Bay, producing an uncontrollable source of pollution. The overflow from sanitary sewers tends to hydraulically overload storm sewers that, in turn, cause problems of surcharging and possible property damage. Excess water from storm sewers discharged into sanitary sewers contributes to overflowing downstream and bypassing at the treatment plant. Each of these problems involves the discharge of pollutants to the Bay.

It would be impossible to enumerate and describe all of the intersystem overflows that exist in the East Bay Area, simply because they are

unknown. However, several types of intersystem overflows are described in Appendix III.

As described earlier, the sewer system in the East Bay Area is extremely complex because it was originally designed as a combined sewer system, and this was later converted to separate sanitary and storm sewer systems. In some areas, the sanitary sewers were designed and constructed specifically for conveying domestic, commercial, and industrial waste waters. In other areas, the sanitary sewer system is a converted combined sewer, complicated by the construction of parallel or relief sewers which were required because of unexpected growth in the service area and excessive infiltration rates. Also, with increasing infiltration rates through poor joints, the need for relief at critical locations in the sewer system became mandatory.

Having limited funds, the various maintenance personnel did their best to relieve the problem of overflowing sewage. Various forms of bypasses were connected from the sanitary sewers to nearby storm sewers that could carry the additional extraneous flow. Each new intersystem bypass or overflow added to the problem, however, until it reached the proportion that exists today. Similar bypasses and overflows were constructed on the storm sewer lines to discharge into oversized sanitary sewers, and these contribute to the problem of system overflows and bypassing of the plant.

Another condition that contributes to overflows is the connection of subsurface drains to sanitary sewers. In areas where groundwater or perched water causes difficulties in roadbed stability, a system of perforated pipes is placed in the ground to drain away the water. These underdrain systems normally discharge into storm sewers; however, in several cases, they were connected into the sanitary sewer system.

If the discharge of pollutants to the Bay is to be stopped by treating or otherwise handling the flow in the sanitary sewer system, the overflows into the storm sewers must be eliminated. If they are not eliminated, the storm sewers will operate as combined sewers, hence their flow may also require treatment prior to discharge into the Bay. Because of the large sums of money that have been spent to date for separating sanitary and storm sewers, it would seem logical that every effort should be made to take advantage of that separation. This would mean elimination of all overflows from sanitary sewers into storm sewers, so that storm water can be discharged into the Bay without treatment.

SYSTEM OVERFLOWS

System overflows occur at locations along the interceptor sewer at points where the trunk sewers are intercepted. The problem associated with these overflows is that of discharging untreated sewage to the Bay during periods of wet weather. The following discussion concerns the purpose of these overflows and their design features.

When the water pollution control plant was constructed in 1951, assurances were obtained from the contributing cities that there would be complete separation of storm and sanitary sewers. For this reason the interceptor sewer and treatment facilities were designed large enough to handle only the expected sanitary and industrial sewage flow, plus a reasonable allowance for infiltration. This peak design flow was 291 mgd. No allowance was made for large quantities of extraneous flow into the sanitary sewer system.

Five combined sewers, serving large areas at the time of the construction of the interceptor sewer, were allowed to be connected, with the understanding that they would also be converted to sanitary sewers as soon as possible. These sewers were connected to the interceptor by way of diversion structures which would direct all of the dry weather flow into the interceptor and divert storm water diluted sewage into the original outfall sewers that discharge directly to the Bay waters. These diversion structures were designed to permit a flow equal in rate to the design dry weather flow to discharge into the intercepting sewer. The flows beyond this were to continue along the original outfall and into the Bay. The existing outfalls were fitted with tide gates to prevent Bay waters from entering the interceptor sewer. Each diversion structure includes a gate that is shut to bypass all or any part of the storm water flow through the existing outfall sewer, thus bypassing the sewage treatment facilities.

In addition to the five diversion structures, three overflow structures were constructed on the interceptor sewer. These are fitted with tide gates to permit sewage to overflow automatically when the intercepting sewer becomes surcharged; no slide gates are involved. Figure 32 shows the approximate locations of all overflow and diversion structures along the intercepting sewer.

WATER POLLUTION CONTROL PLANT BYPASS

There are two basic problems caused by extraneous flows at the water pollution control plant: 1) operation and maintenance problems caused by large quantities of silt and grease which will be discussed here, and 2) bypassing of raw sewage to the Bay, which will be evaluated in Section 9.

Shortly after the water pollution control plant was placed in operation, it became evident that rainfall resulted in abnormally high flow rates together with large quantities of silt and floatable grease. The deposition of large quantities of silt damages the sedimentation basin equipment, and floating grease accumulates in the outfall and related structures.

SILT PROBLEMS

The greatest problem was caused by large amounts of fine sand and silt (particles with diameters of less than 0.2 millimeter) being carried

into the plant. This sand and silt was not removed in the grit chambers, which were designed to remove only particles with diameters greater than 0.2 millimeter. During storms, the fine sand and silt collected in the sedimentation basins in sufficient quantity to cover the sludge collectors completely and to stop the mechanism. After storms when the flow had returned to normal, it was necessary to dewater the channels one at a time and to remove the excess sludge by hand before the chain and flight collectors could be restarted. In addition, pumping sand-laden sludge wore out sludge pump impellers, resulting in high maintenance costs.

When large volumes of sand-laden sludge are pumped into the sludge digesters, they must be frequently taken out of service for cleaning. This problem is both expensive and detrimental to the overall operation of the plant. For this reason sand-laden sludge was bypassed around the digesters and to the outfall for disposal to the Bay.

To eliminate the operational difficulties produced by the large quantities of silt, a bypass was constructed to divert the total wet weather flow around the sedimentation basins. The basis for operating the bypass channel was the objective of a substantial study by the staff of the water pollution control plant (1).

The purpose of the study was to define the point at which bypassing should begin. Based upon laboratory tests and field measurements of both dry and wet weather flows, a value for the electrical resistance of the waste water above which the bypass gate would be opened was set at 1,350 ohms. This represented about a twofold dilution of the sewage, which averaged about 800 ohms. A resistance of 1,350 ohms was equivalent to a plant flow rate of approximately 120 mgd.

Operating experience has confirmed the validity of the study because only acceptable amounts of fine sand and silt are collected in sedimentation basins with flow rates of less than 120 mgd. In actual practice it is this flow rate, not the resistance measurement, that is used to gage the correct opening time for the bypass gates. Use of this criterion involves judgment on the part of the operator; if it appears that the maximum flow rate might only slightly exceed 120 mgd, the bypass gates may not be opened.

Obviously, if improvements could be made in handling or removing the large volumes of sand and silt that are carried into the plant, the bypassing of raw sewage around the sedimentation basins during wet weather could be stopped, or at least substantially reduced. A study of this solution to the problem of bypassing will be undertaken in Sections 10 and 11.

GREASE PROBLEMS

Large volumes of grease also represent a problem at the plant, particularly in the outfall system. During bypassing, the grease passes around

the sedimentation basins and tends to float at both the effluent pumping station and at the outfall transition structure. Grease also clings to the wall of the outfall sewer and accumulates until it begins to break loose in chunks. These chunks of grease, although broken up to a large degree by turbulence in the outfall, contribute to the visible floating material during the rainy season. Cleaning and disposing of the grease that accumulates in the effluent pumping station and in the transition structure requires a substantial amount of personnel time. For example, during the months of January and February 1969, grease and other floatable matter was removed from the outfall transition structure on six different occasions requiring a total of 108.5 man-hours.

If bypassing could be stopped, the problems caused by grease accumulation in the outfall would be reduced. A large portion of the grease would then be removed in the sedimentation basins where provision has been made for its removal.

INTRASYSTEM OVERFLOWS

Four kinds of problems are associated with intrasystem overflows: 1) public health, 2) property damage, 3) nuisance, and 4) rodent control. These problems are caused by overflows at manholes or other openings and broken sanitary sewers.

PUBLIC HEALTH

The concern of public health is the transmission of water-borne diseases. Sewage is known to carry many of the microbial agents of diseases, including typhoid fever bacteria and other members of salmonellae, the viruses of poliomyelitis and hepatitis, and various other intestinal parasites (2,3). Normally, ingestion of infective material is necessary to transmit these diseases. In the case of infectious hepatitis, the infective material may be either fecal matter or blood from an infected person, both of which find their way into the sanitary sewer system.

Although most epidemiological cases involve contaminated water supplies, there still remains the possibility of ingestion as a result of improper personal hygiene coupled with contact with infected matter. This possibility is the source of the public health problem involved with intrasystem overflows. One of the pleasures of children is wading or otherwise playing in rainwater that has collected in street gutters. This in itself is probably not harmful, but if that gutter water contains the remains of sewage discharged from a nearby manhole and if proper hygiene is not practiced, there is the risk of ingestion of contaminants during subsequent eating or drinking.

Fortunately, the probability of contacting a water-borne disease is not high, as indicated by the infectious hepatitis data presented in Tables 19 and 20. Table 19 shows the age distribution for cases of hepatitis in California during 1962. The number of cases for children between one and nine years of age was 875, only about 20 percent of the

TABLE 19

AGE DISTRIBUTION OF INFECTIOUS HEPATITIS
IN CALIFORNIA, 1962

Age	Cases
Under 1	9
1-4	163
5-9	712
10-14	525
15-19	503
20-24	689
Over 25	1,935
Total	4,536

Source: Safety Committee, California Water Pollution Control Association (3).

TABLE 20

MONTHLY DISTRIBUTION OF INFECTIOUS HEPATITIS
IN CALIFORNIA, 1962

Month	Cases
January	487
February	410
March	440
April	390
May	460
June	287
July	319
August	351
September	298
October	470
November	327
December	297
Total	4,536

Source: Safety Committee, California Water Pollution Control Association (3).

total. The evidence does not appear to indicate a disproportionately high occurrence of cases among children. Table 20 shows the monthly distribution for the same data. The six winter months from October through March (50 percent of the calendar year) had 54 percent of the cases. Hence there does not seem to be a significant increase in occurrence during the rainy season. Nevertheless, the possibility of infection exists, and good community hygiene is necessary in controlling or reducing the possibility.

Fortunately, also, the distribution of the locations of the intrasystem overflows indicates that many of them take place in areas involving other than residential land uses. Figure 32 shows the reported locations of commonly occurring overflows from manholes throughout the Oakland and Berkeley systems. The data were collected from maintenance personnel and records from these two cities. These are locations where overflows occur consistently and are to be expected during each substantial rain. No doubt overflows occasionally occur in other locations if storm conditions are right or if some additional problem occurs downstream. Similar data were not collected for the four other cities in the study area; however, the following quote from an engineering report by M. Carleton Yoder, consulting engineer on the Piedmont sewer system, indicates similar problems:

> Other sewers, while not actually overflowing during the past winter (1962-63), had indications that they operated under surcharged conditions during storms. The danger here is that storms of greater intensity than those of last winter could, conceivably, result in overflow of these sewers. Also, surcharging of some sewers caused backing up of sewage into the basement of homes. In several locations, 8-inch sewers run directly into 6-inch sewers causing overflow to the surface and/or surcharging of the line (4).

PROPERTY DAMAGE

A review of Figure 32 and the above quote indicates that property damage is very likely a problem associated with intra-system overflows. The damage can arise not only from sewage backing up into basements, but also from sewage overflowing from sewers located in sideyard and rear-lot easements. These areas are difficult for maintenance personnel to reach.

Claims against the cities for property damage caused by overflowing or backed-up sewage indicate the scope of this problem. Berkeley officials indicated that about four or five such claims are made each year; Oakland officials indicated about 25. The dollar value of these claims was not readily available. Even if it were available, the value would be misleading because much of the clean-up work resulting from overflows on private property is done by city maintenance crews, and their costs would not be included in the claims. Also, inconveniences are caused in some cases where claims are not made.

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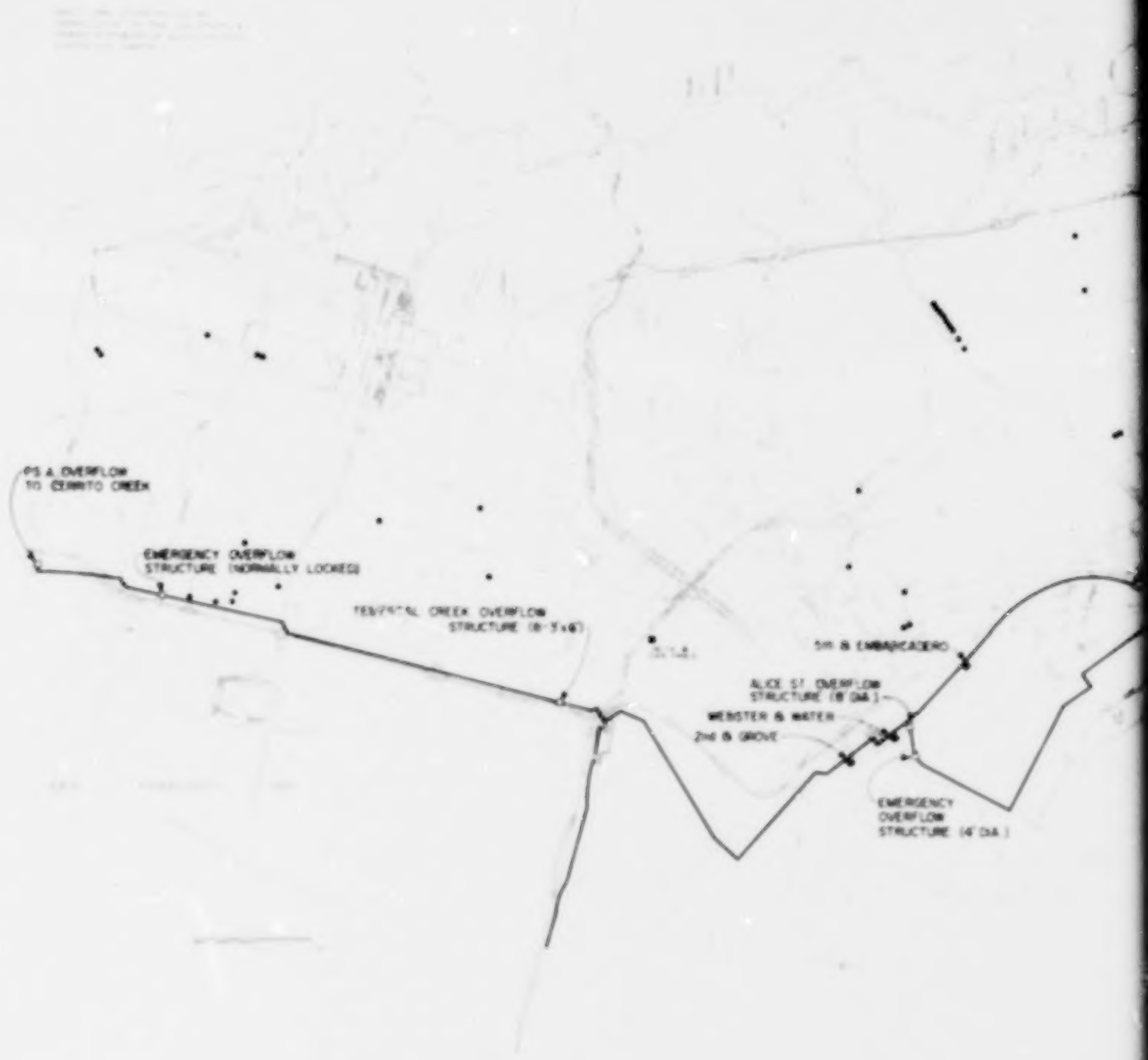
PS A OVERFLOW
TO CERRITO CREEK
EMERGENCY OVERFLOW
STRUCTURE (NORMALLY LOCKED)
STRUCTURE (8'-3x6')
5th & EMBARCADERO
ALICE ST. OVERFLOW
STRUCTURE (8' DIA.)
WEBSTER & WATER
2nd & GROVE
EMERGENCY
OVERFLOW
STRUCTURE (4' DIA.)

(126)

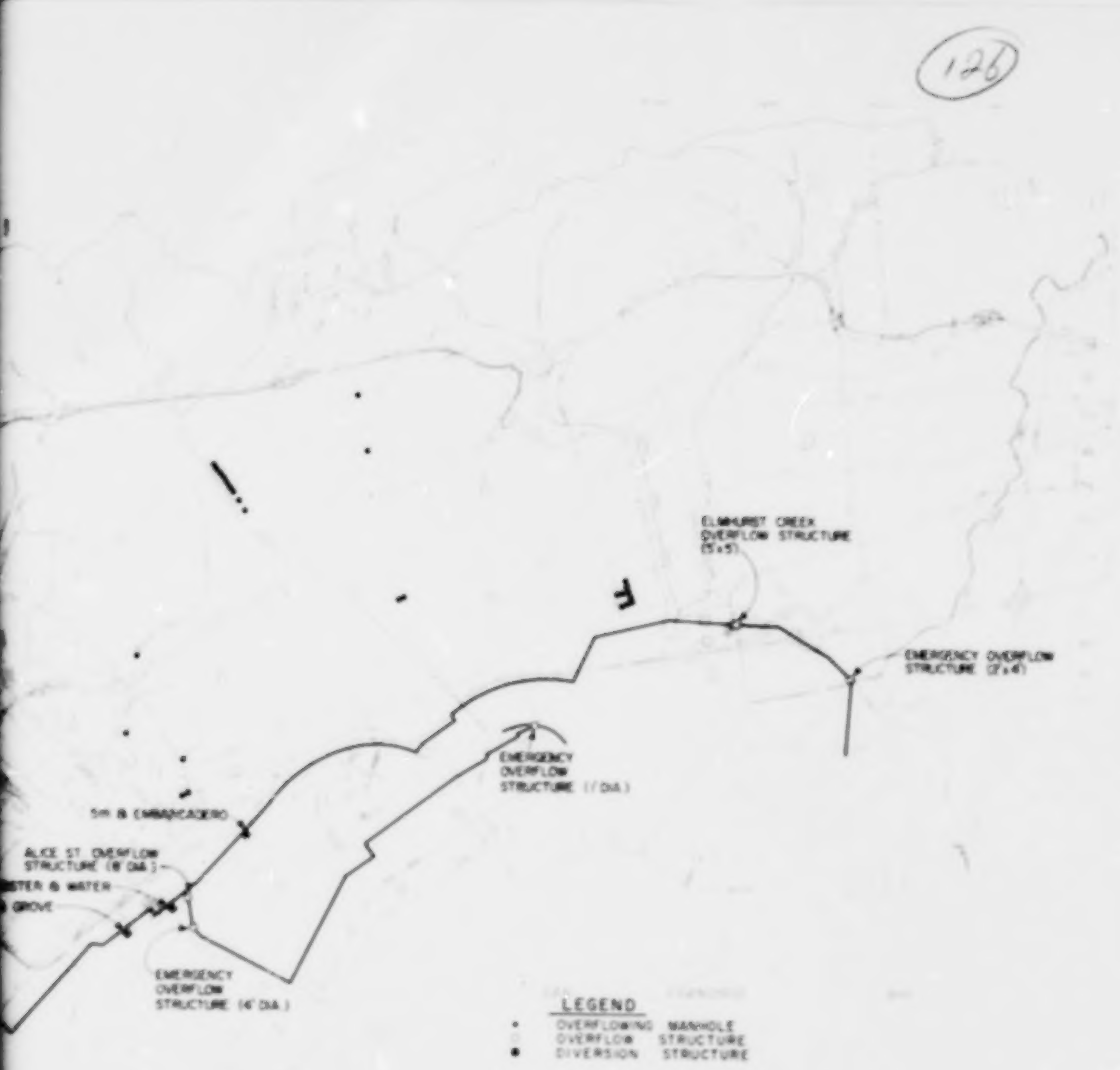

FIG. 32 LOCATIONS OF INTERCEPTOR OVERFLOW AND DIVERSION STRUCTURES AND OF COMMONLY REPORTED MANHOLE OVERFLOWS IN OAKLAND AND BERKELEY

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NUISANCE

The problem of nuisance considerations is obvious. No one appreciates the idea of sewage flowing from manholes and along the street gutters, even when it is diluted with extraneous flows. Knowledge of its presence is repulsive.

An example of the reaction that may be found was a situation in Oakland. Shortly after completion of the Kaiser Center, overflowing sewage was detected in the gutters around the building during rainstorms. A demand was immediately made for correction of the problem and persisted until the condition was alleviated.

RODENT CONTROL

The appearance of rats is very often an indication of a problem in the sanitary sewer system. An opening in the system may offer rats an access to sewage as a source of water, or may permit rats to escape the sewer during an extermination program.

An annual program of rodent extermination in the sewer system is conducted by the local health departments. Rat poison is introduced at selected locations in the sewer system, and the resulting appearances of dead rats are reported by residents. The health department then investigates and recommends corrective action.

Figure 33 shows the distribution of the appearances of dead rats and the reported types of sewer problems in the six East Bay cities for 1967. The data are divided on the basis of census tracts for Oakland, Piedmont, and Emeryville; the data for Berkeley, Alameda, and Albany are for each city as a whole.

The reports revealed that approximately 78 percent represented broken sanitary or building sewers; 6 percent, stopped or plugged sewers; 14 percent, seeping sewage; and 2 percent, flooded basements. Although the distribution of findings was not available from Berkeley, it was assumed that it was similar to that in other areas. These data indicate both the widespread and the general nature of the failures associated with sanitary sewers.

It should not be concluded that these reports represent all of the cases because many are not reported. Only those broken sewers that are associated with the appearance of rats are indicated by these reports.

Because house connections are often broken or plugged, it can be concluded that they may not only contribute to rat infestation but may also cause property damage or nuisance during periods when the sewers are surcharged with extraneous flows. The data further indicate that house connections can contribute to the problem of infiltration.

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FIG. 3
ASSOCIA

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TYPES OF PROBLEMS FOUND UPON INVESTIGATION OF REPORTS

	TOTAL * REPORTS	BROKEN SEWER	STOPPED SEWER	SEEPING SEWAGE	FLOODED BASEMENTS
NO.	197	101	8	17	2
%	100 * *	79	6	14	2

* INCLUDES [illegible] REPORTED PROBLEMS FROM BERKELEY WHICH WERE NOT DEFINED

* * ASSUMES DISTRIBUTION OF PROBLEMS IN BERKELEY ARE SIMILAR TO THAT IN OTHER AREAS

FIG. 33 DISTRIBUTION OF REPORTS ON RAT APPEARANCES AND ASSOCIATED TYPES OF SEWER PROBLEMS IN EAST BAY CITIES, 1967

SOURCE: COMPILATION OF DATA FROM ALAMEDA COUNTY HEALTH DEPARTMENT AND BERKELEY AND ALBANY CITY HEALTH DEPARTMENTS

SECTION 8

WATER QUALITY OBJECTIVES

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SECTION 8

WATER QUALITY OBJECTIVES

SELECTED OBJECTIVES

To determine the effects of overflows on the quality of the Bay water, water quality objectives must first be established. In this study, the objectives adopted by the San Francisco Bay Regional Water Quality Control Board were used except where they were deficient in numerical limits. These objectives were part of Resolution No. 67-30, "Water Quality Control Policy-Tidal Waters Inland from Golden Gate," and will be hereinafter referred to as the Regional Board Objectives (1). In order to establish numerical limits, the objectives recommended by the San Francisco Bay-Delta Water Quality Control Program (2) were used. These objectives have been subsequently considered by the Regional Board for adoption, but no action has been taken to date. These objectives are hereinafter referred to as the Bay-Delta Objectives.

The objectives were evaluated to determine 1) which parameters could be selected for use in assessing the effects of overflows, and 2) what rational values could be used for the selected parameters. Three water quality parameters were selected: dissolved oxygen (DO), floatable materials, and coliform bacteria. These parameters are consistent with the data gathering program of the study and can be used with available data and evaluation procedures for making a reasonable assessment of pollution effects. The selected objectives are given in Table 21. The following discussion elaborates further on why these parameters were selected (and why others were not) and describes the rationale used in establishing numerical values.

THE REGIONAL BOARD AND BAY-DELTA OBJECTIVES

The excerpt below from Resolution No. 67-10 indicates the Regional Board's awareness of the problem of overflows and indicates their policy toward its solution.

> This Regional Board has considered the discharge of untreated sewage into public streets and into waterways from overloaded sanitary sewer systems during periods of rainfall the same as any other sewage discharge as defined in the California Water Code. The Regional Board has prescribed requirements or prohibited these discharges on a case-by-case basis; however, these problems have generally been given a lower priority over other pollution problems because of their intermittent nature (1).

TABLE 21

SELECTED WATER QUALITY OBJECTIVES FOR
EVALUATING EFFECTS OF OVERFLOWS

Parameter	Unit	Value
(1) Dissolved oxygen	mg/L	Minimum of 5.0
(2) Floatable materials		
On water surfaces	g/sq m	Maximum of 0.02
On shore areas	g/sq m	Maximum of 0.3
(3) Total coliform bacteria	MPN/100 ml	Maximum of 1,000*

*In a 30-day period the total coliform MPN must not be greater than 1,000/100 ml more than 20 percent of the time. No samples are to contain more than 10,000 coliform per 100 ml when verified by another sample within 48 hours.

Sources: (1) and (3) Regional Board Objectives (Ref. 1).
(2) Bay-Delta Objectives (Ref. 2).

On the basis of the Regional Board statement, the determination of the effects of overflows on the Bay waters may be made using the water quality objectives normally used for continuous discharges. In the past, this type of discharge has been given a lower priority because of the more pressing problems of continuous discharge. At the present time, however, consideration of requirements is imminent for both intermittent discharges and overflows.

Table 22 lists the two sets of water quality objectives that were considered applicable for the East Bay waters. Column 1 lists the Regional Board Objectives, which apply at the outer limit of the rising waste plume or beyond a limited dilution area as determined by the Regional Board on a case-by-case basis. If the dilution zone is small in area, these objectives could apply to the water being discharged, although their intent is for measurement of receiving water quality. Several parameters are not quantified under the Board's present policy. It would therefore be difficult to determine the adverse effects of the pollutional matter discharged to the Bay using these parameters. The only parameters with specific limits are DO, coliform bacteria, and pH.

Column 2 summarizes the Bay-Delta Objectives, which were recommended for Water Quality Zones 3 and 4 (hereafter referred to as Zones 3 and 4), shown on Figure 34.

More of the parameters listed under the Bay-Delta Objectives have numerical limits. In addition, two new parameters--biostimulation and relative toxicity--are introduced. Neither of these could be used

TABLE 22

REGIONAL BOARD AND BAY-DELTA WATER QUALITY OBJECTIVES
APPLICABLE TO SAN FRANCISCO BAY VICINITY OF EBMUD, SPECIAL DISTRICT NO. 1

Water Quality Parameter	(1) Regional Board Objectives	(2) Bay-Delta Objectives (Zones 1 and 4)
Dissolved oxygen	Minimum of 5 mg/l; when natural factors cause lesser concentrations, then controllable water quality factors shall not cause further reduction...	Median ≥ 7.5 mg/l 95 percentile ≥ 6.5 mg/l
Floatable materials	None other than of natural causes.	Floatable materials shall not be present in concentrations which cause interference with identified beneficial uses, particularly esthetic enjoyment and recreational uses. (Suggested guide - Max. concentration of oil materials not to exceed 0.12 g/sq m on water and 0.3 g/sq m on shore areas.)
Coliform bacteria	In most cases, 7957 and 7958, Title 17 of California Administrative Code in designated areas.*	Median ≤ 70 MPN/100 ml 95 percentile ≤ 230 MPN/100 ml
Biostimulants	--	Limit flows of primary or secondary effluents to 1980 levels. Transport flows above projected amounts from Zone or provide tertiary treatment to remove 90 percent of the biostimulants with discharge locally.***
Relative toxicity	--	40 ml/l****
Toxic materials	None present in concentrations deleterious to beneficial uses; none at levels which render aquatic life or wildlife unfit for human consumption.**	--
Temperature	See Appendix Table III-1.	See Appendix Table III-1.
Turbidity (Clarity or transparency)	No significant variation beyond present natural background levels.**	Maintain present transparency pending studies to evaluate the effects of changes on algal populations and other organisms.

Table II (concluded)

Color	No significant variation beyond present natural background levels.**	--
Bottom deposits	None other than of natural causes.**	--
Nutrients	None causing deleterious or abnormal growths.	--
Radioactivity	See Appendix Table III-1.	--
pH	7.0 - 8.5	5 percentile ≥ 7.5 95 percentile ≤ 8.5
Oil or materials of petroleum origin	None floating in quantities sufficient to cause an iridescence, or none suspended or deposited on the substrate at any place.	--
Odors	None other than of natural causes at any place.	None objectionable which are man-induced and/or controllable.
Pesticides	No pesticide or combination of pesticides shall reach concentrations found to be deleterious to fish or wildlife at any place.**	No quantitative objective - recommend that the manufacture, sale and use of persistent pesticides, particularly the chlorinated hydrocarbons, be regulated to eliminate insofar as possible their entry to the aquatic environment.
Methylene blue active substances	--	median ≤ 0.5 – mg/l 95 percentile ≤ 0.75 – mg/l

*In a 30-day period the total coliform MPN must not be greater than 1,000 per 100 ml more than 20 percent of the time. No samples are to contain more than 10,000 coliform per 100 ml when verified by another sample within 48 hours.

**The water quality objective will generally apply at the outer limit of the rising waste plume or beyond a limited dilution area as determined by the Regional Board on a case-by-case basis.

***Design criteria pending studies to develop water quality objectives for biostimulation.

****Relative toxicity concentration calculated from amounts and locations of discharges assuming concentration of discharged relative toxicity = 1,000 ml/l.

Sources: (1) San Francisco Bay Regional Water Quality Control Board, Resolution No. 67-37 (1967) and Resolution No. 69-6 (1969).
(2) San Francisco Bay-Delta Water Quality Control Program, final report, preliminary edition, March 1969.

FIG. 34 WATER QUALITY ZONES OF THE SAN FRANCISCO BAY-DELTA AREA

because they were developed and recommended after the initiation of this study and adequate data were not accumulated. Nevertheless, they will be defined and discussed in this section.

For this study, the water quality objectives for DO and coliform bacteria were selected from the Regional Board Objectives, and the objective for floatable materials was selected from the Bay-Delta Objectives.

SELECTED PARAMETERS

DISSOLVED OXYGEN

In Column 1 of Table 22 the Regional Board Objective for DO content is listed as not less than 5 mg/L, whereas the Bay-Delta Objective (Column 2) is for a median value of not less than 7.5 mg/L with a 95 percentile value of not less than 6.5 mg/L. Although the Bay-Delta Objective appears to be substantially higher than the Regional Board Objective, it seems to be more in line with the actual DO content of the major water masses in the Bay. Therefore, the Bay-Delta Objective is an attempt to establish and maintain the present DO level.

In reality, the two objectives may not be as different as they first appear. The Bay-Delta Objective establishes a median value for DO together with a lower limit, whereas the Regional Board Objective establishes only a minimum value. Because most of the available data provide mean values of DO rather than extreme values, the Bay-Delta Objective is more applicable to this investigation.

The Regional Board Objective of a minimum of 5.0 mg/L DO was selected for use in this study because it is actually in effect, whereas the Bay-Delta Objective is only a recommendation at this time.

FLOATABLE MATERIALS

In the Bay-Delta Program it was assumed that the objectionable portions of the floatable materials are related in some way to oil and grease. Based on this assumption, the maximum concentration of oily material was recommended not to exceed 0.02 gram per square meter on water surfaces or 0.3 gram per square meter on shore or beach surfaces. Obviously, not all floating materials contain grease or can be indicated by measuring oil and grease. Also, very little data are available to support this approach or to aid in its application. However, because oil and grease tend to aggregate other solid matter and form an objectionable float or scum, this objective for floatable material seems to be a reasonable approach. For this reason and because extensive data on oil and grease were collected during this study, this objective was selected for evaluating the pollutional effects of overflows.

COLIFORM BACTERIA

The Regional Board Objective for coliform bacteria is based on total coliform concentrations. The coliform test involved in this objective is the presumptive test as described in Standard Methods (3). This objective for coliform presently meets the California Health Department requirements for water contact sports (4).

The Bay-Delta Objective for coliform bacteria is based on fecal coliforms. The determination for fecal coliforms requires an additional step beyond the standard presumptive test, involving incubation of media innoculated from the positive tubes in the presumptive test for 24 to 48 hours at an elevated temperature, usually between 43 deg C and 48 deg C (5). This objective meets the U.S. Public Health Service criteria for shellfish harvesting areas (6).

Because all of the coliform analyses made as part of this investigation were by the presumptive test, the Bay-Delta Objective could not be used. Therefore, the Regional Board Objective will be used in this study as a means of defining the effects on the Bay waters from overflows.

OTHER PARAMETERS

BIOSTIMULANTS

The term biostimulants is intended to encompass all of the nutrient material that may be discharged to a receiving water. Nutrients are those materials that increase the rate of eutrophication or aging of the water mass. The concept of the term recognizes that eutrophication is not the result of any single nutrient, such as phosphorus or nitrogen, but involves the combined effect of all nutrients, including those trace elements necessary to sustain biological life and reproduction. It was presented in the Bay-Delta Program report with an explanation of limitations and the need for extensive study and definition. The concept of biostimulation provides a means for assessing and comparing the total relative amounts of biostimulants or nutrients discharged from various sources in the future.

The recommended water quality limits set forth in Table 22 for biostimulants in Water Quality Zones 3 and 4 require limiting flows of primary or secondary effluents to 1980 levels. Therefore, biostimulation resulting from overflows in the East Bay Area is assumed to be not critical at this time, but it will become more critical as the projected flow rates for 1980 are approached.

RELATIVE TOXICITY

The relative toxicity of a particular waste stream is defined as its flow rate divided by the 48-hour median tolerance limit (TL_m) expressed as a fraction. (The TL_m is the concentration of a waste, expressed as a percent or decimal fraction, which kills one-half of the test fish in

the given exposure time). For example, if the effluent flow rate from the EBMUD Water Pollution Control Plant averaged 70 mgd and had a TL_m of 66 percent, the relative toxicity would be 106 mgd (70 mgd/0.66 = 106 mgd). Thus, the parameter takes into account both the amount and toxic strength of the waste. The Bay-Delta Program report states:

> Field studies have demonstrated a relationship between the calculated concentrations of relative toxicity in the southern region of the Bay and the observed effects on the benthos (bottom animals). These studies show that an adverse effect is discernible when the ratio of dilution water to relative toxicity is less than 25 to one (40 milliliters of relative toxicity per liter of dilution water). The benthos must be considered as a critical ecological element since these animals represent 85 percent of the total protein in the San Francisco Bay waters. They are an integral and necessary part of the food chain of fish and it is almost certain that any conditions adverse to benthic biota will also harm fish (2).

Relative toxicity represents a new approach to evaluating the effects of waste waters, and substantial research effort will be required to define more precisely the causes and effects. In addition, more information is needed on the dispersion patterns of waste discharges so that areas of effect can be delineated. When more information is known in the future, it is probable that relative toxicity will be used extensively as a measure of adverse effects of waste discharges.

MISCELLANEOUS

Several other water quality parameters were considered for evaluating the effects of overflows on the Bay, including temperature, pH, methylene blue active substances, pesticides, odors, color, and bottom deposits. Either because of the limited state of knowledge, lack of quantitative values, or lack of available data, these water quality parameters could not be used. It should be noted, however, that on February 13, 1969, the Regional Board adopted specific numerical temperature objectives for the water quality zone inland from the Golden Gate (Resolution No. 69-6). These temperature objectives, presented in Appendix Table III-1, supersede those adopted as part of Resolution No. 67-30. They recognize both a seasonal and geographical variation in temperature levels and do not apply to tide flats or other very shallow portions of the system. The waters in these areas may be naturally warmer than the main water masses.

It is interesting to note the temperature data taken by EBMUD from their north and south interceptors during the months of January, February, and March 1969, when substantial amounts of the season's rain fell. The minimum temperatures were substantially lower than those recorded during the other months of the year. The average

combined minimum temperature for these three months was approximately 57 deg F, which is only slightly higher than the 95 percentile temperature limit of 56 deg F for Water Quality Zone 3. Therefore, temperature should not be a controlling parameter although overflows occur along the shoreline where minimum dilution is provided.

SECTION 9

EVALUATION OF EFFECTS OF OVERFLOWS ON SAN FRANCISCO BAY

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SECTION 9

EVALUATION OF EFFECTS OF OVERFLOWS ON SAN FRANCISCO BAY

This section is devoted to an evaluation of the effect of overflows on each of three selected water quality objectives. For this purpose, pollution will be assumed to exist if the computed values of the water quality parameters exceed the selected water quality objectives.

It should be understood that an extensive sampling of the Bay to determine the background quality and to determine the effects of overflows accurately was beyond the scope of this investigation. The intent of this section is to provide an order-of-magnitude type of analysis by using existing available data, together with procedures supported by previous work reported in the literature. Results and findings developed here are used in Sections 10 and 11 to evaluate the various alternative solutions to overflows that are available.

A summary of the quantity and quality of overflows and their frequency is presented in Table 23. This table includes the data that were developed in Sections 5 and 6 and that are used in the computations in this section.

CONCLUSIONS

The evaluation produced the following significant conclusions:

- Results of the computations made on each of the three selected parameters indicated that only floatable materials and coliform bacteria exceed the selected objectives during or just after overflows.

- Dissolved oxygen (DO) is the least affected of the three parameters that were evaluated. Although the DO is depressed by overflows, the combined effect of overflows and storm sewer discharges is required to produce conditions that violate the objectives. Available background DO data show that this condition does occur during the rainy season, but usually only within the months of January, February, or March (see Figures 36 and 37).

- The duration or persistence of floatable materials in the near-shore waters was found to be six and two days for Zones 3 and 4, respectively. These time periods were based on the time duration required for the floatable grease concentrations to return to within 0.01 gram per square meter of the selected objective which was 0.02 gram per

TABLE 32

SUMMARY OF DATA USED TO EVALUATE EFFECTS OF OVERFLOWS ON SAN FRANCISCO BAY

Item	Zone 3	Zone 4	Water Pollution Control Plant Bypass
Quantity:			
Number of storms per year producing overflows and bypasses	11.5	11.5	11.5
Volume of overflow or bypass per storm, MG	62.5	28	68
Distribution of plant bypass, percent	50	50	--
Quality:			
BOD*, mg/L	370	105	135
Grease, mg/L	105	21	40
Coliform bacteria, MPN/100 ml	3.7×10^8	1.5×10^7	--

*5-day Biochemical Oxygen Demand.

square meter. The floatable materials were estimated to affect 22 square miles of water surface area, considering both Zones 3 and 4.

- Coliform bacteria discharged during overflows were found to produce concentrations above the selected objective for approximately 2.6 and 1.5 days after each overflow event for Zones 3 and 4, respectively. Therefore, the objective for coliform bacteria would be exceeded in the East Bay waters as a direct result of overflows for about 23 days per year. The computations considered only bacteria discharged in numbers beyond those normally discharged during dry weather. This was done because the background count in the near-shore waters usually equals or slightly exceeds the selected objectives. Persistence was based on the time for the bacteria either to disperse in the Bay or to die-away until their concentrations returned to within 10 MPN per 100 ml of the normal background counts. The area of effect was assumed to be equal to that for floatable materials.

- Coliform bacteria concentrations and their time of persistence beyond the selected objectives would also be substantially increased by storm sewer discharges. Although the evaluation of storm sewer contributions is an important consideration, it was assumed that their effects would be lessened by implementing the recommendations of this report.

- From the preceding findings it was concluded that any alternative solutions for alleviating the pollutional problems by treating the extraneous flow prior to overflow should be directed at reducing the grease and the coliform bacteria. Because of the inherently high DO levels during the rainy season of the year, organic material from overflows alone will normally not depress the DO levels below the selected objective.

Other pertinent conclusions are as follows:

1. Average DO levels are well above the minimum objective of 5.0 mg/L during the rainy season.

2. Localized and short-lived DO levels were noted to fall below the minimum objective during the rainy season.

3. The low values of DO found during the rainy season were considered to be the result of the combined effects of overflows and storm sewer discharges.

4. The background concentration of floatable materials in the East Bay waters presently equals or slightly exceeds the objective of 0.02 gram per square meter.

5. The allowable floatable grease emission was found to be 310 pounds per day into Zone 3 and 375 pounds per day into Zone 4.

6. A relationship between grease concentration and percent of floatable grease was derived from grease removal data in the sedimentation basin at the plant.

7. From existing background coliform bacteria data, it was found that the objective is presently exceeded in the shoreline waters between Albany and North Alameda and throughout the Oakland Inner Harbor but is being met along the Bay side of Alameda.

8. The East Bay waters offshore from Albany, Berkeley, and Emeryville normally meet the objective for coliform bacteria and likely exceed it only because of a combination of overflows and storm sewer discharges during the rainy season.

9. The East Bay waters offshore from southern Alameda normally meet the coliform objective. Again, the limitations are similar to those waters offshore from Berkeley and Emeryville.

DISSOLVED OXYGEN

An adequate level of oxygen dissolved in water is necessary for protection of aquatic life. The fish are usually among the first to be affected by a reduction in DO levels. Next are the smaller organisms that provide the food supply for the fish. Lower oxygen levels may result in a change in the type of bacterial population both in the water and in the bottom sediments. At zero DO levels the anaerobic organisms, which include such organisms as the methane bacteria, would predominate. Fortunately, this condition, which is normally accompanied by the presence of gas bubbles at the surface of the water, has not prevailed since the construction and operation of the interceptor sewers and the water pollution control plant.

DO may be the oldest parameter used for measuring pollution in receiving waters. Reduced DO levels indicate the discharge of excessive amounts of organic matter to the receiving waters. This causes the bacterial population to increase and its combined metabolism to exert a demand on the DO quantities in the water, resulting in a reduction of the oxygen resources. Similar effects can be noted in the Bay waters. For this reason, the DO objective, although set to protect or enhance aquatic organisms, will be used as a measure of the allowable organic material that may be discharged to the Bay via overflows.

Limited data are available for determining the background DO concentrations in the East Bay waters to which the overflows are added. However, by using data collected during the 1963-64 "Comprehensive Study of San Francisco Bay" made by the University of California (1), and 1967-69 receiving waters data from EBMUD, an estimate of background levels was made. Dispersion data developed in the Bay Model by the Corps of Engineers and BOD data collected during this investigation were used to estimate the effects of overflows on the background DO levels. BOD measurements indicate the amount of oxygen required to stabilize the organic material discharged, hence provide a means for estimating the effect on the oxygen resources in the receiving waters.

BACKGROUND DISSOLVED OXYGEN LEVELS

Because there is a background of organic and nutrient materials in the Bay waters, a substantial population of algae is maintained at all times. Algae and various other forms of chlorophyll-containing plankton require the presence of sunlight to carry on their metabolic processes. These organisms produce oxygen by photosynthesis during the daylight hours but consume oxygen at night, thereby causing the oxygen level in the receiving water to vary throughout the day.

Figure 35 shows the indicated diurnal variation of DO, derived from data taken during the daylight portions of April 10 and 11, 1969, near Doolittle Bridge at the mouth of San Leandro Bay. The points shown on this graph are the average values based on two days of sampling. It may be seen that the DO levels were quite high, averaging 9.4 mg/L. Using the water temperatures and chlorosity levels prevalent in that area, the DO level at saturation was 9.0 mg/L; therefore, the average DO content was approximately 7 percent above the saturation level.

Figure 35 indicates that there is a difference of approximately 2 mg/L between the high and low DO values in the Bay waters. This variation is significant and should be recognized in the conduct of Bay sampling programs.

Figures 36 and 37 represent the variation in DO level for four different locations in the East Bay waters during the 1968-69 fiscal year. The mean DO in the EBMUD outfall area remained well above the objective of 5.0 mg/L during the rainy season. The mean DO levels along the south shore (Alameda) varied from a low of 6.1 up to a high of 9.4 and down to 8.7 in that same period of time. It will be noted from Figure 36 that the mean DO levels during the dry season are substantially below the selected objective DO. A similar occurrence appears on Figure 37 for the north shore stations (Emeryville and Berkeley). This low DO level during the late summer season may be attributable to three factors: higher salinity values, higher water temperatures, and greater organic loading on the Bay resulting from cannery discharges.

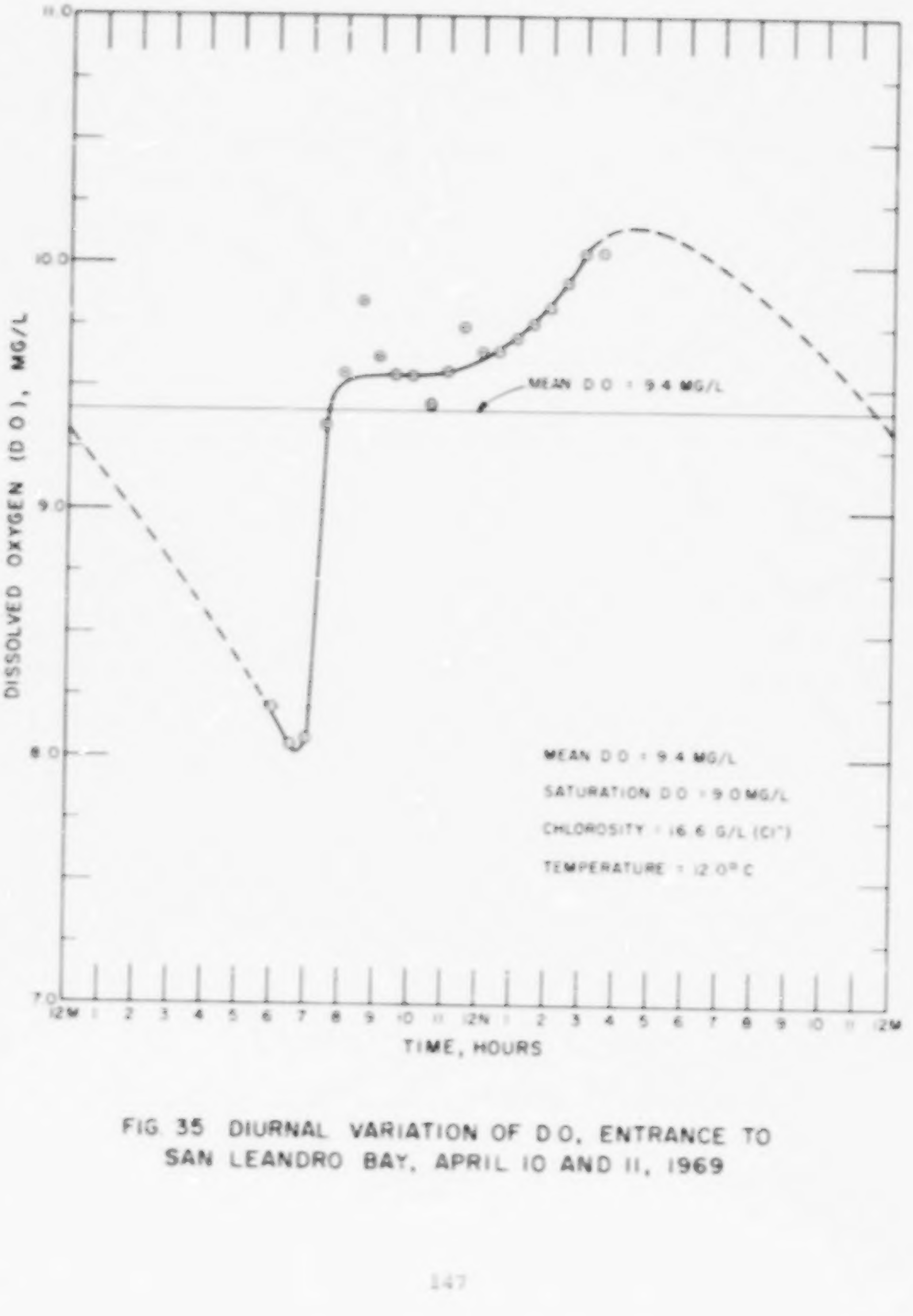

FIG. 35 DIURNAL VARIATION OF D.O., ENTRANCE TO SAN LEANDRO BAY, APRIL 10 AND 11, 1969

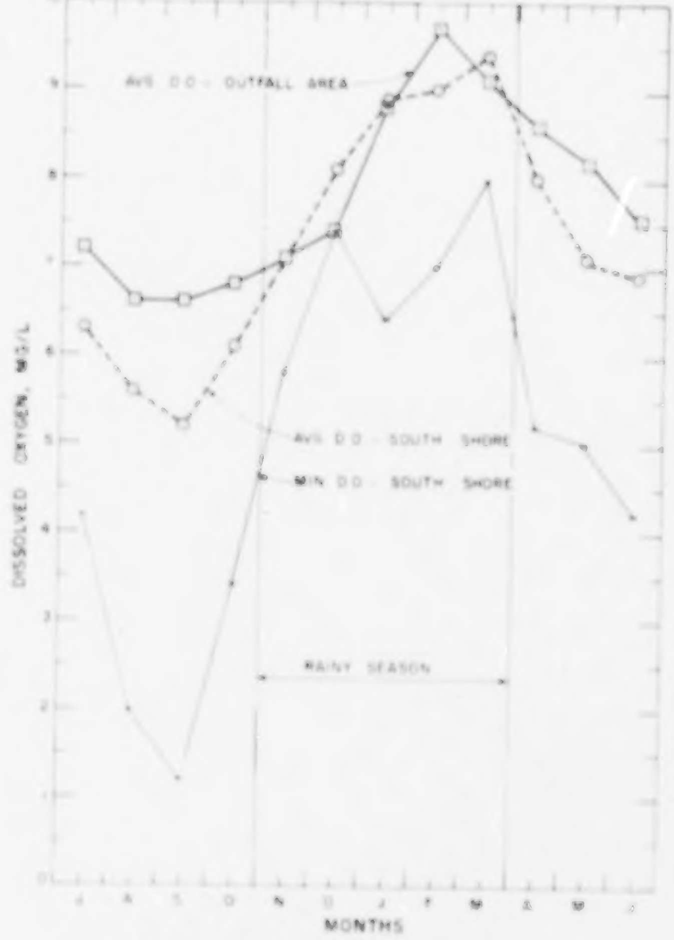

FIG. 36 MONTHLY VARIATION IN D.O. CONCENTRATION, EAST BAY AREA, OUTFALL AND SOUTH SHORE, 1968-69

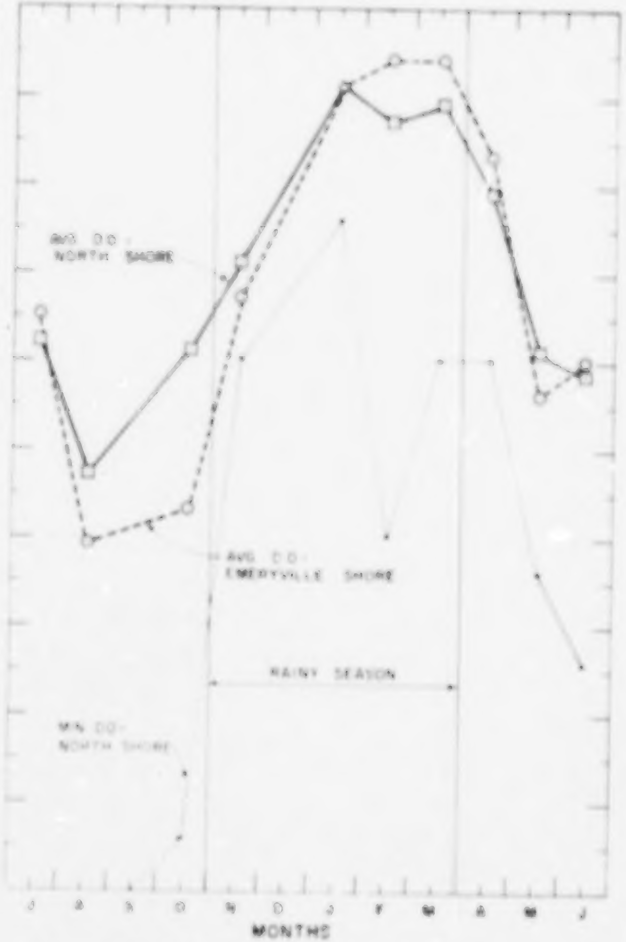

FIG. 37 MONTHLY VARIATION IN D.O. CONCENTRATION, EAST BAY AREA, EMERYVILLE AND NORTH SHORE, 1968-69

SOURCE: MONTHLY REPORTS, WATER POLLUTION CONTROL DIVISION, EBMUD SPECIAL DISTRICT NO. 1, 1968-1969

temperature and chlorosity vary throughout the year in a manner similar to that shown in Figure 38. Both of these curves were developed by combining data collected during 1963-64 for the north and central San Francisco Bay areas.

While discharges from canneries in the East Bay Area do affect the DO levels, the canning season is concluded by the time the rainy season starts, so cannery discharges should produce no effect on the receiving waters to overshadow or compound the problem of overflows. The only possibility would be an abnormally early rain from a high intensity and substantially long duration storm.

It will be noted that the minimum DO levels are substantially below the mean values during the rainy months of January and February. These differences probably indicate the effect of intermittent discharging of oxygen demanding materials to the Bay waters during these two months.

ESTIMATES OF DISSOLVED OXYGEN DEPRESSION CAUSED BY OVERFLOWS

Will the organic material discharged by overflows alone cause a depletion of the DO resources to a level below that prescribed by the water quality objectives? To answer this question the following steps were necessary:

1. The total mass of organic material (BOD) discharged to the receiving water by overflows and bypasses was computed. The BOD is then distributed throughout the receiving waters by tidal action or dispersion.

2. The dispersion characteristics for the north shore and south shore waters were defined.

3. The procedure was developed for equating biological decomposition to DO depression.

4. DO depression was computed, considering both dispersion and biological decomposition.

Mass of Organic Material Discharged

The concentration of BOD in the overflows and in the bypassed sewage will be assumed equal to the BOD values measured during this investigation at Pump Stations A and H and at the water pollution control plant. As developed in Section 6, the average frequency of overflows will be taken as 11.5 per year evenly spaced over the rainy season and the amount of sewage discharged at each overflow occurrence will be 62.5 MG into the Oakland Inner Harbor, 28 MG into the north shore waters, and 68 MG into the outfall dispersion area. This latter number is based on bypassing sewage at the rate of 168 mgd for a period

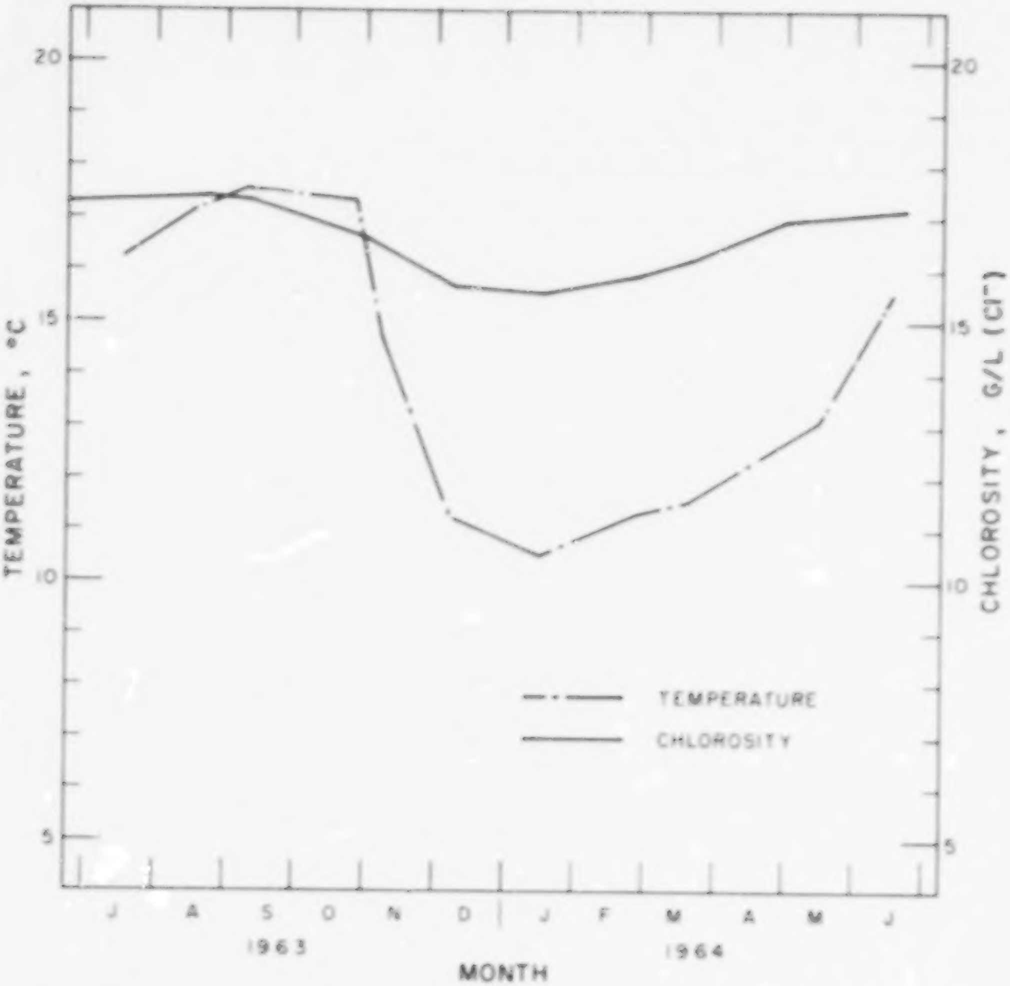

FIG. 38 SEASONAL VARIATION OF TEMPERATURE AND CHLOROSITY, NORTH AND CENTRAL AREAS, SAN FRANCISCO BAY, 1963-64

SOURCE: COMPOSITE OF MEAN TEMPERATURES FOR CENTRAL AND NORTH SAN FRANCISCO BAY, STORRS, ET AL (1)

of 9.6 hours at the plant via the outfall. It will be assumed, on the basis of float studies made in the "1941 Sewage Report" (2), that 50 percent of the bypassed flow would enter Zone 3 and 50 percent would enter Zone 4. It will also be assumed that 30 percent of the BOD discharged into each zone will enter the respective near-shore waters and will effectively combine with the BOD discharged in the overflows. This latter assumption is admittedly weak; however, the 30 percent figure seems to be conservative and will serve to indicate the overall effect of overflows and bypassing until more exact relationships can be determined in the future by either mathematical modelling or tests in the Bay Model.

Dispersion Characteristics

Dispersion data in the East Bay waters are limited to dye studies made by the Corps of Engineers in the San Francisco Bay Model. A comprehensive study requiring dye release points at several locations throughout the Bay Area was made for the U.S. Public Health Service (3). Figures 39 and 40, taken from these studies, show dye dispersion contours for releases near Government Island, Oakland Inner Harbor, after one and three tidal cycles, respectively. In addition to these two conditions, data were developed for up to 40 tidal cycles.

Figure 41 is a plot of dye concentration versus time at several stations within the Oakland Inner Harbor and the San Leandro Bay system. As seen on the plot, there is no distinct dispersion of the dye from the Inner Harbor until almost ten cycles have elapsed. At that point a definite dispersion rate seems to occur. Although there are several points of overflow into Oakland Inner Harbor, the fact that there is no distinct dispersion for ten tidal cycles after the overflow permits all discharges to be evaluated as a single discharge. Considering the Oakland Inner Harbor and San Leandro Bay as a single entity, an average concentration was selected between the first and tenth tidal cycles equal to 400 ppb per gram of dye released. The concentration of dye after ten cycles seems to follow the relationship:

$$C = 1.82 \times 10^{5} t^{-2.66} \text{ for values of } t \geq 10$$

where C is in units of ppb/g.

This relationship, together with an allowance for decay, was used to compute the concentration of BOD in the receiving water at any time after the occurrence of an overflow within the Oakland Inner Harbor-San Leandro Bay system.

The Bay Model studies do not include dye release for a point along the north shore, so the dispersion characteristics had to be estimated. Richmond was nearest the dye release point for which data were available. Within one tidal cycle, substantial dye reached the waters offshore from Berkeley and Emeryville. Figure 42 shows the dye concentrations after three tidal cycles. This figure is typical of the dye

NOTES:

1. CONTOURS REPRESENT DYE CONCENTRATIONS IN PARTS PER BILLION PER GRAM OF DYE RELEASED (PPB/G).
2. BASE TEST WAS CONDUCTED FOR PUBLIC HEALTH SERVICE AND MARITIME ADMINISTRATION - AEC FLUSHING STUDIES.
3. FRESH WATER INFLOW AT CHIPPS ISLAND = 16000 CFS.

DYE TRACER:

CONCENTRATION = 1000 PPM
VOLUME = 6 LITERS
QUANTITY = 6.0 GRAMS

LEGEND

▲ INJECTION STATION
Ⓡ AUTOMATIC FLUOROMETER & RECORDER
x T-5 FEET BELOW WATER SURFACE
x M- MIDDEPTH
x B-5 FEET ABOVE CHANNEL BOTTOM

FIG 39 DYE DISPERSION CONTOURS AT HIGHER HIGH WATER SLACK, INJECTION AT OAKLAND INNER HARBOR, END OF CYCLE I

SOURCE: CORPS OF ENGINEERS, (3)

NOTES

1. CONTOURS REPRESENT DYE CONCENTRATIONS IN PARTS PER BILLION PER GRAM OF DYE RELEASED (PPB/G)
2. BASE TEST WAS CONDUCTED FOR PUBLIC HEALTH SERVICE AND MARITIME ADMINISTRATION – AEC FLUSHING STUDIES.
3. FRESH WATER INFLOW AT CHIPPS ISLAND = 16000 CFS

DYE TRACER:

CONCENTRATION = 1000 PPM
VOLUME = 6 LITERS
QUANTITY = 6.0 GRAMS

LEGEND

▲ INJECTION STATION
Ⓡ AUTOMATIC FLUOROMETER & RECORDER
x T-5 FEET BELOW WATER SURFACE
x M-MIDDEPTH
x B-5 FEET ABOVE CHANNEL BOTTOM

FIG. 40 DYE DISPERSION CONTOURS AT HIGHER HIGH WATER SLACK, INJECTION AT OAKLAND INNER HARBOR, END OF CYCLE 3

SOURCE: CORPS OF ENGINEERS, [3]

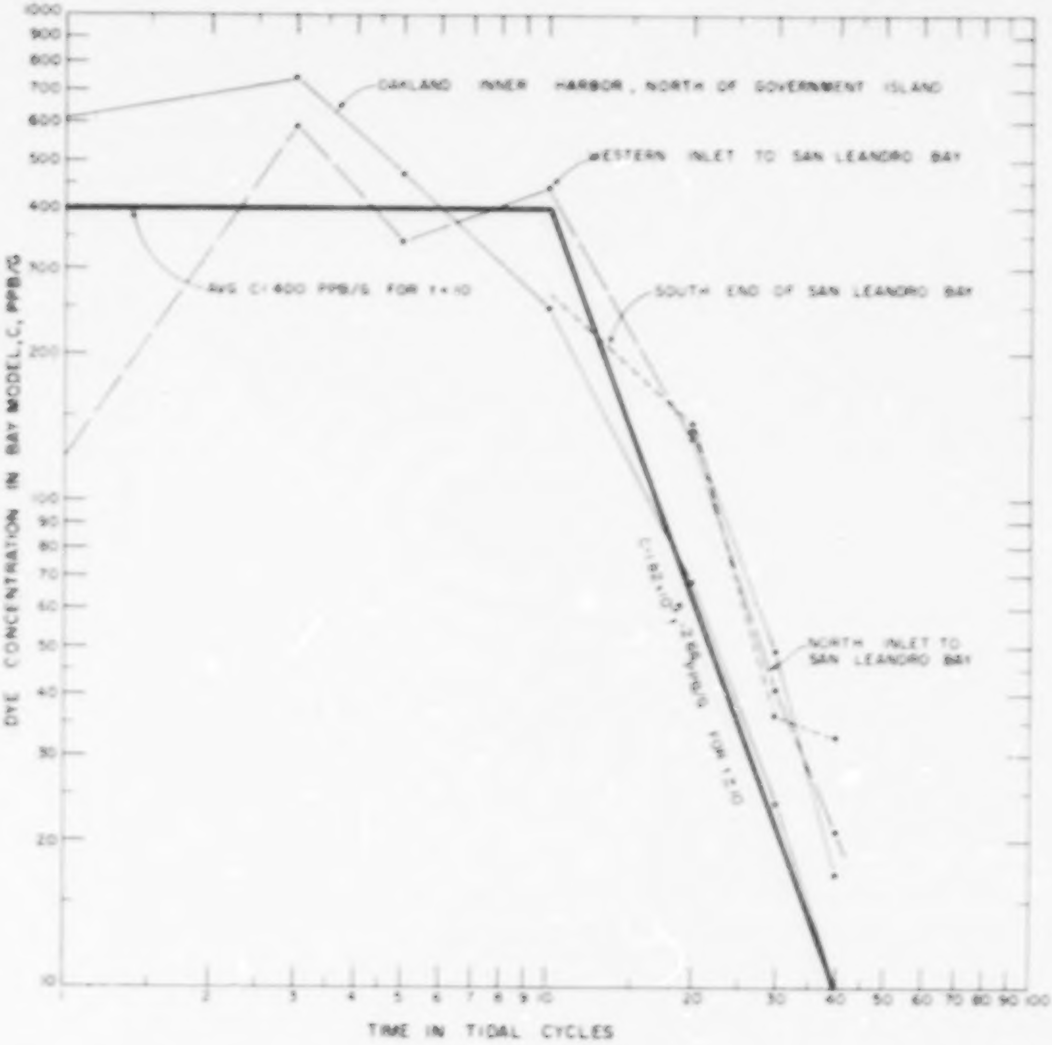

FIG 41 ESTIMATED DISPERSION RELATIONSHIPS FOR POLLUTANTS WITHIN OAKLAND INNER HARBOR AND SAN LEANDRO BAY

NOTES:

1. CONTOURS REPRESENT DYE CONCENTRATIONS IN PARTS PER BILLION PER GRAM OF DYE RELEASED (PPB/G).
2. BASE TEST WAS CONDUCTED FOR PUBLIC HEALTH SERVICE AND MARITIME ADMINISTRATION – AEC FLUSHING STUDIES.
3. FRESH WATER INFLOW AT CHIPPS ISLAND = 16000 CFS.

DYE TRACER:

CONCENTRATION = 1000 PPM
VOLUME = 6 LITERS
QUANTITY = 6.0 GRAMS

LEGEND

▲ INJECTION STATION
Ⓡ AUTOMATIC FLUOROMETER & RECORDER
x T-5 FEET BELOW WATER SURFACE
x M-MIDDEPTH
x B-5 FEET ABOVE CHANNEL BOTTOM

FIG. 42 DYE DISPERSION CONTOURS AT HIGHER HIGH WATER SLACK, INJECTION AT PT. RICHMOND, END OF CYCLE 3

SOURCE: CORPS OF ENGINEERS, (1)

concentration distribution over the next 37 cycles. The greatest concentration is seen near the shore, with the lesser concentrations toward the deeper portions of the Bay, similar to what one would expect if the dye had been released along the shoreline.

Figure 43 is a plot of dye concentrations at several points in the north shore waters versus time. After reaching a point of maximum concentration, it can be seen that lines drawn through the data points were reasonably parallel, indicating a general trend in dispersion rate. Because these data points represented only a small portion of the dye actually released at Point Richmond, they could not represent the actual concentration of dye if it had been released along the Berkeley or Emeryville shoreline. However, if an initial concentration could be established, the same rate of dispersion could be reasonably used.

To compute the initial dye concentration that might occur if released along the shoreline, it was assumed that the dye concentration at the end of one tidal cycle would be zero at a point three miles from shore (west end of Berkeley Pier) and would be maximum at the shoreline. Hence, the average dye concentration would occur at a point one and one-half miles offshore. Using the soundings presented on a USC&GS map of the San Francisco entrance, the volume of water in the prototype between the San Francisco-Oakland Bay Bridge on the south and Point Isabel on the north and a width of three miles was found to be 48×10^8 cubic feet. One cubic foot in the Bay Model is equal to 1×10^8 cubic feet in the prototype; therefore, the corresponding volume of water in the model at this location would be 48 cubic feet or 1,460 liters. Injecting one gram of dye would produce a concentration of 685 ppb per gram of dye in this volume. Therefore, the estimated concentration of dye after one tidal cycle at a point located one and one-half miles offshore would equal 685 ppb per gram of dye released. Plotting this value on Figure 43 and drawing a line through it with a slope equal to that of the lines drawn through the data points previously plotted, yields the estimated dispersion rate for dye if released along the Berkeley or Emeryville shoreline. The equation is:

$$C = 685\ t^{-1.13} \text{ for values of } t \geq 1.0$$

where C is in units of ppb/g.

Because of the gross nature of the dispersion equation, no attempt could be made to evaluate single or individual overflows. In this zone the need to evaluate single overflows is not great, because the major overflow does occur at a single point.

<u>Procedure for Computing Dissolved Oxygen Depression</u>

The equation for BOD concentration in the receiving water from overflows (considering dispersion only) was developed in the Bay Model

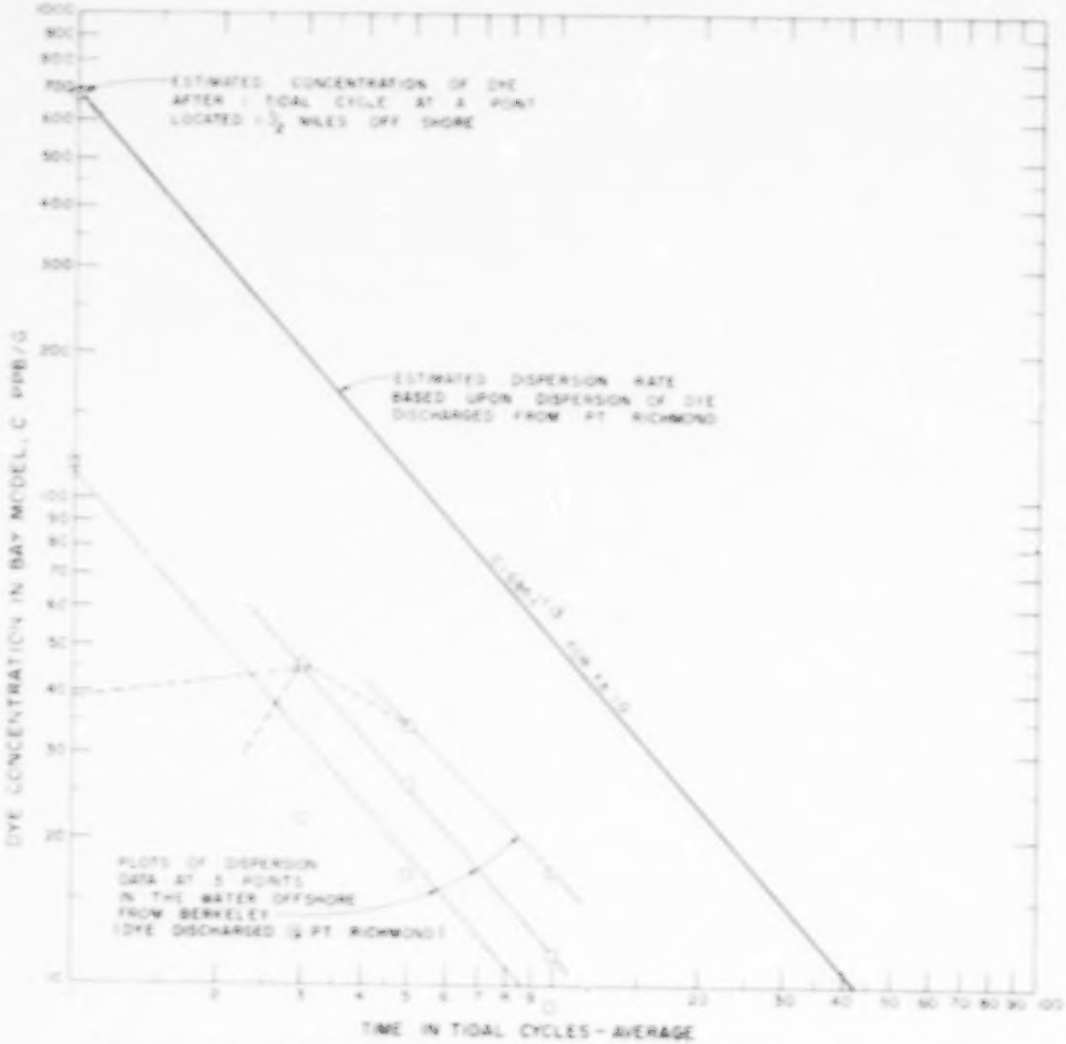

FIG. 43 ESTIMATED DISPERSION RELATIONSHIPS FOR POLLUTANTS APPROXIMATELY 1-½ MILES OFF THE EMERYVILLE, BERKELEY, ALBANY SHORELINE

SOURCE: CORPS OF ENGINEERS [illegible]

studies (3) and is as follows:

$$L_5 = C\ (\text{ppb/g}) \times 4.54 \times 10^{-6} \times \text{BOD discharged } (1{,}000\ \text{lbs})$$

where L_5 = BOD (5 day, 20 deg C), mg/L

C = dye concentration at the point in question, ppb/g of dye released

4.54×10^{-6} = factor for converting model units to prototype units

BOD discharged = 8.33 x overflow volume (MG) x BOD (mg/L)

To determine the total BOD concentration in the receiving waters, the background BOD concentrations must be added to the above computed concentrations. Values of receiving water BOD in the outfall area during 1968-69 varied from less than 0.1 mg/L to 1.9 mg/L. Values for the months of November through March were distinctively lower than those for the other months of the year, averaging about 0.35 mg/L. For purposes of these computations, a value of 0.5 mg/L of background BOD was assumed.

Having the expression for evaluating dispersion on BOD concentrations in the receiving water, the Streeter-Phelps equation may be used concurrently to estimate the resulting oxygen depression:

$$D_t = \frac{k_1 L_a}{k_2 - k_1}\left(10^{-k_1 t} - 10^{-k_2 t}\right) + D_a 10^{-k_2 t}$$

where D_t = oxygen deficit at reference point at time t, mg/L

D_a = initial DO deficit in the receiving water immediately after mixing with the waste water, mg/L

L_a = ultimate first stage BOD, 1.46 x L_5, mg/L

L_5 = BOD (5 day, 20 deg C), mg/L as computed by expression for dispersion

t = time, days after overflow

k_1 = deoxygenation constant, 0.10 at 20 deg C, days^{-1}

k_2 = reaeration constant, 0.24 at 20 deg C, days^{-1}

T = temperature, deg C.

The Streeter-Phelps formulation assumes that BOD removal consists only of aerobic biological degradation, and that reaeration is simply absorption of atmospheric oxygen. Also inherent in this equation is the assumption that the rate of removal of oxygen by benthic demand and plant respiration is equal to the rate of addition of oxygen by photosynthesis.

To be usable, the deoxygenation and reaeration constants, k_1 and k_2, must be adjusted for temperature. This adjustment may be made using the following equation:

$$k_T = k_{20} a^{T-20}.$$

It was assumed that the value of a is constant and is equal to 1.047 for the deoxygenation constant and 1.016 for the reaeration constant.

It should be noted that in the foregoing equation the value for L_a is a variable. This variability takes place because dispersion occurs in the receiving waters, hence reducing the BOD concentration with time. However, if the value of L_a is recomputed for each time period, using the dispersion relationships as determined above, the variability can be accounted for.

Computation of Dissolved Oxygen Depression

To illustrate the magnitude of the oxygen deficit resulting from the average overflows, a series of calculations was made for each of the two water quality zones for the average overflow conditions. The results are tabulated in Table 24. The typical conditions selected for these computations are notes at the bottom of the table.

Table 24 indicates that organic materials discharged by overflows alone do not normally cause depression of DO levels below those permitted by the selected objective. This conclusion does not indicate that overflows, together with storm sewer discharges (during storms), will not depress the oxygen levels below those recommended. In fact, Figure 37 shows that DO values below 5.0 mg/l occasionally occur during the months of January, February, and March. Also, storms that are larger than normal might produce unacceptable DO levels from overflows for a short time, particularly near the shore in the immediate vicinity of the discharges.

FLOATABLE MATERIALS

Floatable materials may consist of any substance or objects that will float on the surface of the water. They may be derived from many sources, including spilled motorboat fuel, bilge pumping, storm sewer discharges, overflows, and sewage treatment plant effluents. Floating materials are most objectionable when they are foreign to their surroundings. For example, floating globules of grease or matrixes of

TABLE 24

EFFECTS OF OVERFLOWS ON DISSOLVED OXYGEN
IN THE NEAR-SHORE WATERS OF WATER QUALITY ZONES 3 AND 4

Water Quality Zone 3						
Time after beginning of overflow and bypass, days	1	2	3	4	5	10
Initial DO deficit*, mg/L	0.50	0.50	0.50	0.50	0.50	0.50
Imposed DO deficit*, mg/L	0.37	0.28	0.22	0.18	0.14	0.06
Total DO deficit, mg/L	0.87	0.78	0.72	0.68	0.64	0.56
Saturation DO*, mg/L	9.50	9.50	9.50	9.50	9.50	9.50
Median DO, mg/L	8.63	8.72	8.78	8.82	8.86	8.94
Objective:						
Minimum DO, mg/L	5.0	5.0	5.0	5.0	5.0	5.0
Water Quality Zone 4						
Time, days	1	2	3	4	5	
Initial DO deficit*, mg/L	0.50	0.50	0.50	0.50	0.50	
Imposed DO deficit*, mg/L	0.09	0.07	0.05	0.04	0.03	
Total DO deficit, mg/L	0.59	0.57	0.55	0.54	0.53	
Saturation DO*, mg/L	9.50	9.50	9.50	9.50	9.50	
Median DO, mg/L	8.91	8.93	8.95	8.96	8.97	
Objective:						
Minimum DO, mg/L	5.0	5.0	5.0	5.0	5.0	

*Computation based on chlorosity of 15 grams per liter and temperature of 10.5 deg C.

grease and other particulate matters are very objectionable in the water environment. The presence of sticks, leaves, and other natural detritus, although undesirable, is not nearly as objectionable as is the presence of oil and grease (hereafter referred to as grease). Because of this factor and because a measurable parameter was needed, grease was selected as a measure of floatable materials. As discussed previously, the water quality objective for floatable materials was based on a measure of the mass of the floating grease in units of grams per square meter.

Very few data are available to determine the background grease concentrations in the East Bay waters to which the grease from overflows is added. Also, very few data are available to substantiate the selected permissible, floatable emissions in the East Bay Area. However, by using certain assumptions made in the Bay-Delta Program report, data from the EBMUD Water Pollution Control Plant, and dispersion characteristics developed in the Bay Model by the Corps of Engineers, estimates were made of the background grease concentrations relative to the water quality objectives and the increases in grease content immediately after the occurrence of an overflow.

BACKGROUND GREASE LOADING

Grease normally entering the water pollution control plant is partly removed in the sedimentation basins either by floating or by settling. The grease and other materials that float to the surface, called scum, are removed and disposed of by incineration. The grease that settles is removed from the sedimentation basins together with the sludge and pumped to the digesters where anaerobic digestion takes place. Not all of the grease is broken down during the period of digestion; therefore, grease is discharged to the outfall with the digested sludge. The grease that is not removed in the sedimentation basins together with that from the digesters, passes to the outfall and is discharged to the receiving waters.

Floatable Grease Emission During Dry Weather

Between 1960 and 1967, the average grease concentration in the effluent from the sedimentation basins was 32.9 mg/L and the average flow through the plant was recorded as 74.6 mgd. Therefore, 20,500 pounds per day of grease were discharged to the receiving waters after passing through the sedimentation basins. Furthermore, during this period, an average of 3.00 MG per month of digested sludge with a grease content of 0.5 percent was discharged to the outfall. This additional discharge of grease averages 4,100 pounds per day. Thus, the total amount of grease discharged to the receiving waters from the plant averaged 24,600 pounds per day, and the average concentration of grease in the effluent was 39.5 mg/L, based on an average flow rate of 74.6 mgd.

Obviously, not all of the grease that is discharged to the receiving waters--either in raw sewage from overflows or from the water pollution control plant--rises to the surface as floating material. To provide a means for estimating the fraction of discharged grease that will appear as floating material, an analysis was made of the influent grease concentrations versus removal in the sedimentation basins, together with an analysis of the fraction of grease removed in the form of scum. The results are shown graphically on Figure 44.

The data consist of monthly averages for raw sewage, settled sewage, weight of scum removed, and percent of grease in the scum. Because the data were somewhat erratic, they were grouped into ranges based on raw influent concentrations, and the average values from these ranges were plotted on Figure 44. Curve A describes the overall removal of influent grease in the sedimentation basins. Curve B indicates the fraction (in percent) of the grease that was removed in the sedimentation basins (Curve A) where it appeared as scum or as floatable material. Curve C is the product of Curves A and B so that the percent of raw grease that will float could be read directly. The data seem to indicate that for influent grease concentrations below 30 to 40 mg/L, the majority of grease is removed by floating. Concentrations of influent grease above 40 mg/L tend to be removed by a combination of settling and flotation, with settling predominating in ranges above 40 mg/L.

In order to estimate the amount of floatable grease discharged to the Bay each day, it was assumed that the grease in the plant discharge would settle or float in a manner similar to that of the influent grease depicted in Figure 44. However, certain limiting assumptions were also made. First, only the grease that would be discharged with the digested sludge would react in the manner described in Figure 44. The grease in the sedimentation basin effluent would remain in the suspended state. Second, because salt water is conducive to congealing grease, the percentages of floatable grease indicated by Figure 44 should be increased by 50 percent.

The average concentration of grease in the plant effluent attributable to digested sludge is 6.6 mg/L. Using the nearest influent oil concentration on Figure 44, 10 mg/L, the probable proportion of grease discharged that would rise as floatable matter would be about 18 percent. Increasing this figure by 50 percent, and applying that to the mass of grease discharged, indicated a floatable grease emission of 1,100 pounds per day (0.18 x 1.50 x 4,100).

Assimilative Capacity of the Receiving Waters

Does a discharge of 1,100 pounds per day of floatable grease exceed the water quality objective established for floatable materials? To answer this question, it was necessary to define the assimilative capacity of the receiving waters.

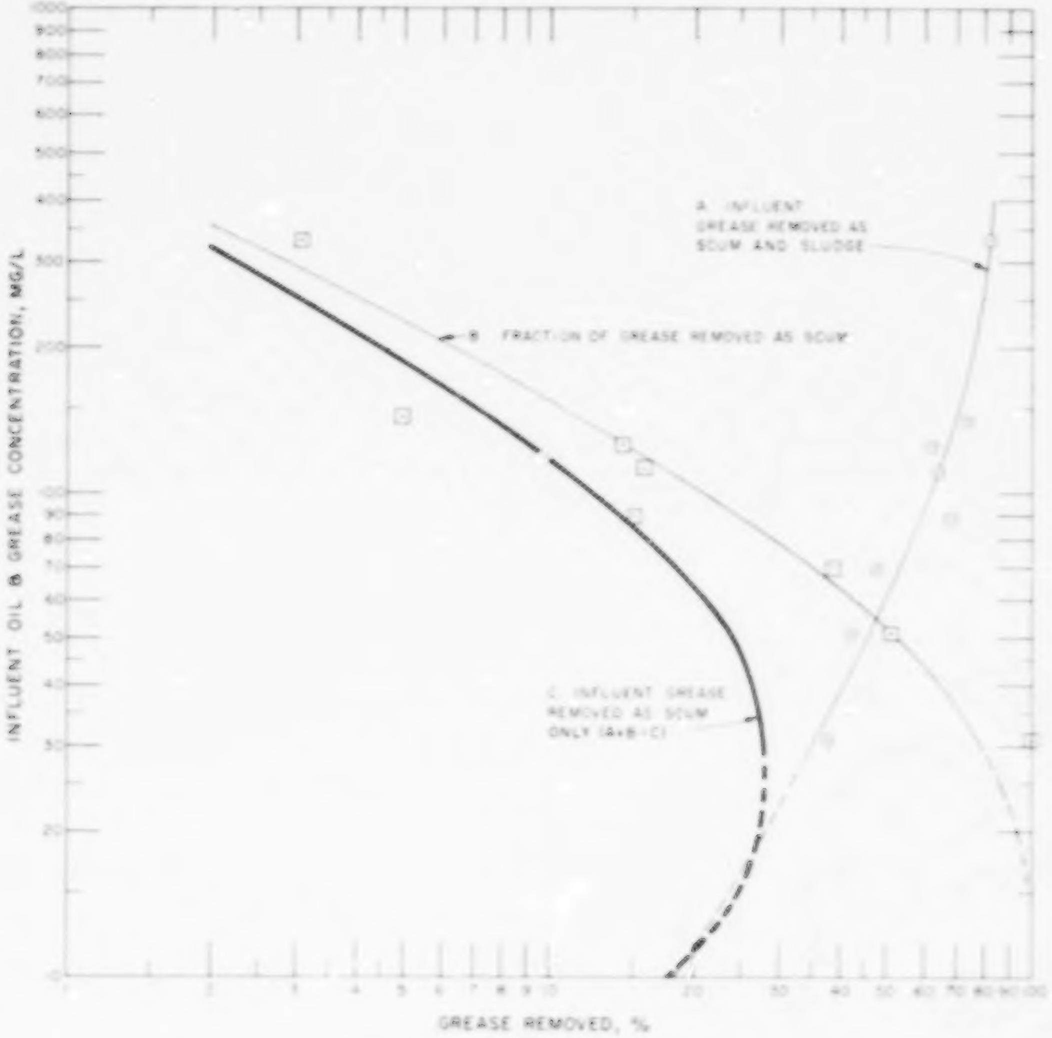

FIG 44 GREASE REMOVED BY PRIMARY TREATMENT VERSUS INFLUENT GREASE CONCENTRATION

SOURCE: BASIC DATA FROM [illegible] WATER POLLUTION CONTROL PLANT ANNUAL REPORTS, [illegible]

In the Bay-Delta Program report, the principal removal mechanism for grease was assumed to be biological oxidation, occurring on the shoreline between the mean low water and mean high water and in the one-half mile of water nearest the shore. It was also assumed that the rate of removal, both from the water and the shore, is 3.5 percent per day. This rate was based on studies of biological oxidation of motor oils in water at a temperature on 10 deg C.

The second removal mechanism was assumed to be the net daily tidal exchange. Between 15 and 20 percent of the tidal prism in San Francisco Bay was estimated to be replaced by new ocean water during each tidal cycle. This is the single most important mechanism by which pollutants are ultimately removed from the Bay.

These two mechanisms were used to estimate the allowable grease load on the receiving waters during dry weather conditions.

On the basis of the previously mentioned assumption that the pollutional materials discharged in the vicinity of the EBMUD outfall would disperse almost equally between the north and south directions, approximately 550 pounds per day of floatable grease would be discharged into the easterly portions of Zones 3 and 4.

Studies were also made for the "1941 Sewage Report" using free-floating bottle floats. Of 54 bottle floats released in the vicinity of the present outfall, 29 were recovered. About 80 percent of those found in Zone 3 were along the shoreline of northern Alameda and the Oakland Middle and Outer Harbors. Obviously, the direction and velocity of the wind during the float studies played a very important role in their distribution, probably more important than the tidal currents. About 65 percent of those floats recovered in Zone 4 were located along the shoreline between Emeryville and Point Isabel; the others were located in remote parts of the central San Francisco Bay.

From the results of these studies it was assumed that tidal currents have a greater effect on floating material in Zone 4 than in Zone 3 when the material is discharged in the vicinity of the EBMUD outfall. Therefore, it was assumed that 80 percent of the floatable material discharged into Zone 3 and 60 percent of that discharged into Zone 4 from the outfall will reach the near-shore waters or shoreline.

Considering only that portion of the East Bay waters lying within Zone 3 (south of the San Francisco-Oakland Bay Bridge), the shoreline available for accumulation of floating materials was estimated to be 20,000 meters in length. Assuming this shoreline is basically three meters in width plus the large areas (mud flats) that are exposed during low tidal conditions, the estimated shore area available for accumulation of grease and its biological oxidation is approximately 6.7 million square meters. This amount includes the mud flats within San Leandro Bay and along the northerly line of Bay Farm Island and the Bay side of Alameda.

Using a projected length of the shoreline and a one-half-mile width of water surface, the water surface area available for assimilation and oxidation of grease was estimated to be 13.6 million square meters. Applying an areal loading of 0.02 gram per square meter on the water surface and 0.3 gram per square meter on the shore between mean high water and mean low water, a maximum allowable accumulation of grease is found to be 5,500 pounds. Assuming a decay rate of 3.5 percent per day for grease on both surfaces together with a tidal exchange rate of 20 percent applied only to grease on the water surface, the maximum allowable floatable grease emission into Zone 3 was found to be 310 pounds per day.

The actual floatable grease emission into the near-shore waters of Zone 3 is about 140 percent of the allowable grease emission (0.80 x 550 x 100/310 = 140%).

Turning to Zone 4, a similar set of calculations was made to determine the permissible load and the daily emission of floatable materials from the East Bay system. Three assumptions were made for these calculations that were different from those in Zone 3: 1) the tidal exchange rate was only 15 percent; 2) the near-shore waters were considered to reach to the westerly end of the Berkeley Pier, or about three miles; and 3) only 60 percent of the floatable materials discharged from the EBMUD outfall would reach the near-shore areas. A lower value for tidal exchange was assumed because more of the north shore area has shallow water and mudflat exposure during low tides. Also, a definable current pattern does not exist in these waters during the ebbing and flooding of the tides.

Float studies made at the end of the Berkeley Pier in the "1941 Sewage Report" showed that, from that point, there was very little movement into or away from the shore during the various tidal positions. For these reasons the near-shore waters were assumed to reach to the end of the Berkeley Pier, or approximately three miles from the north shoreline. The assumption regarding the accumulation of floatable materials discharged in the vicinity of the EBMUD outfall was based on the free-floating bottle study described previously.

On the basis of the above assumptions, the permissible floatable grease emission into the East Bay portion of Zone 4 was 375 pounds per day. The actual load was computed to be 330 pounds per day, which is 88 percent of the permissible loading.

In the Bay-Delta study the overall comparison of present floatable emissions and assimilative capacities for Zones 3 and 4 indicated that the emissions (1965) were 212 percent of the acceptable discharges in Zone 3 and 105 percent of the acceptable discharges in Zone 4. These computations took into account both the eastern and western shores of the zones and all of the major municipal and industrial waste dischargers. It should be noted that floatable grease may also be contributed to Zone 3 by the City of San Leandro, the Oro Loma Sanitary

District, and the Hayward sewage treatment plants, and to Zone 4 by the City of Richmond and the Stege Sanitary District sewage treatment plants. To a much lesser degree, dischargers on the west side of the Bay may also add to the floatable concentrations in the East Bay waters. Of course, the reverse is also true. Having no better data to substantiate assumptions, it will be assumed that the interchange of floatable materials between the east and west shore dischargers is equal and will not substantially change the floatable emissions computed above.

The inherent assumption that other dischargers along the east shoreline would have no effect on the floatable emissions in the near-shore waters of EBMUD, Special District No. 1 may not be applicable. The three dischargers located south of Bay Farm Island may not contribute substantially to the problem because of the natural barrier provided by Bay Farm Island and because the prevailing winds are toward shore, tending to maintain the floatable materials within the one-half mile zone.

This may not be the case in Zone 4 along the north shoreline. Dye studies made in the Corps of Engineers' Bay Model indicate that pollutants will be carried quickly from the vicinity of the Richmond outfall into the waters offshore from Albany, Berkeley, and Emeryville. Also, the discharge from the Stege plant will tend to contribute because it discharges directly into these waters. However, use of this plant will soon be discontinued and the sewage will be pumped into the EBMUD interceptor for disposal through their plant. Therefore, it may be reasonable to assume that the floatable emissions and accumulations in the waters along the north shore are presently equal to the permissible levels of the water quality objective.

Although the floatable emission exceeds the permissible level in Zone 3, substantial quantities of grease and related floatable materials are not presently noticeable in these waters. Therefore, for purposes of this investigation and for evaluating the effects of overflows on the receiving waters, it will be assumed that the present grease concentrations in both Zones 3 and 4 are acceptable during dry weather, but that grease discharged by overflows and bypasses during wet weather will exceed the water quality objective.

ESTIMATES OF QUANTITY, DURATION, AND AREAL DISTRIBUTION OF GREASE FROM OVERFLOWS

It was assumed that no grease would be discharged from the digesters during periods of bypassing. Because the sedimentation basins are out of service during bypassing, no sludge would be pumped to the digesters and no supernatant or digested sludge would be displaced.

The weight of floatable grease discharged in Zone 4 was computed in a way similar to that for Zone 3, except that only 60 percent of the grease discharged from the plant entered the near-shore waters.

Using the procedures outlined for floatable grease emissions during dry weather, the quantity of materials discharged during an average storm condition and expected to float was found to be 12,900 pounds to Zone 3 and 4,600 pounds to Zone 4.

To compute the duration of time that grease concentrations would exceed 0.01 gram per square meter (greater than the objective for floatable material), it was assumed that the grease would be removed by both tidal exchange and biological oxidation. The tidal exchange characteristics were assumed to be equal to those depicted on Figures 42 and 43, which show variations in dye concentrations measured in the Bay Model versus time for both Zones 3 and 4. In Zone 3 the dye concentrations presented on Figure 41 apply only to areas within the Oakland Inner Harbor and San Leandro Bay System. In Zone 4, the concentrations apply at points approximately one and one-half miles offshore between Point Isabel and San Francisco-Oakland Bay Bridge. It was also assumed that grease globules or particles would be distributed throughout the water in a way similar to that portrayed by the dye releases. The distributed grease would then float in quantities proportional to the uniform grease concentration suggested by the dye dispersion data. The floatable fraction would be determined from Figure 44. It was further assumed that biological oxidation would occur at a rate of 3.5 percent per day. These assumptions resulted in the following general equation:

$$F_T = G_o\,(C \times 4.54 \times 10^{-6})(0.305d)(1.5R_f)(1.00 - 0.035)^T$$

where F_T = floating grease concentrations at time T, grams per square meter

G_o = weight of grease discharged in 1,000 pounds

C = 400 ppb/g for Zone 3 for $t<10$

= $1.82 \times 10^5\ t^{-2.66}$ ppb/g for Zone 3 for $t \geq 10$

= $685\ t^{-1.13}$ ppb/g for Zone 4 for $t \geq 1$

d = depth of water at mean high water, feet (approximately 12 feet)

R_f = ratio of floatable grease to total grease discharged (read directly from Figure 44 for the grease concentration in the overflow)

t = time, tidal cycles (approximately 2 per day)

T = time, days

4.54×10^{-6} = factor for converting model units to prototype units.

From the above formulation, the areal grease concentrations resulting from an average wet weather condition, as defined in Section 5, was found to persist for a period of six days in Zone 3 and two days in Zone 4. The area assumed to be affected by floating grease in Zone 3 is equal to the one-half mile of near-shore waters along a projection of the south shore line, which is a total of 7.5 square miles. The area affected by floating grease in Zone 4 is equal to the three-mile width of near-shore waters between Point Isabel and the San Francisco-Oakland Bay Bridge, which is a total of 14.5 square miles. Therefore, the surface area of the East Bay waters that are adversely affected by floatable materials is about 22 square miles (7.5 + 14.5).

The values of areal and temporal floatable grease distribution on the receiving waters should be used, not with a high degree of certainty, but rather with a realization of the nature and number of assumptions necessary. Obviously, the computations could be improved substantially by using a two-dimensional mathematical model for defining the dispersion from both inflows and outflows. Also, additional dye releases could be made in the Bay Model to show the dispersion from the overflows in Zone 4 and from the EBMUD outfall. Additional research is needed to define the fraction of grease discharged that may be expected to float, and also the rate of biological oxidation that may be expected to occur in this environment.

COLIFORM BACTERIA

The main purpose of establishing a water quality objective for bacteria in the receiving waters is to protect the recreational users of that water from enteric diseases. The indicator organisms selected for measuring the safety of the waters for recreational users has historically been the coliform group of bacteria. The Standard Methods (4) definition of this group suggests the test procedures necessary for their identification:

> The coliform group includes all of the aerobic and facultative anaerobic, Gram-negative, nonspore-forming, rod-shaped bacteria which ferment lactose with gas formation within 48 hours at 35 deg C.

The greatest shortcoming of the coliform test is not that it will fail to indicate a state of pollution, but rather that it may indicate pollution where none exists. For this reason more definitive tests are commonly being proposed for water quality objectives, but with each degree of refinement, the cost of monitoring the water increases.

Because the usual testing procedure for the coliform group involves multiple-tube fermentation procedures and not a direct enumeration of organisms, a statistical procedure is commonly used for estimating the number of coliform organisms that existed in the sample. This estimated number of organisms or the most probable number is usually identified by the abbreviation MPN. It should be understood that this is

merely an index to the number of coliform bacteria which, more probably than any other number, would give the same results as a more direct laboratory examination. It is not an actual enumeration of the coliform bacteria in any given volume of sample.

Most of the data that were available for evaluating the effect of overflows on the coliform counts in the Bay were from the presumptive coliform test, with the exception of those made by the EBMUD Water Pollution Control Plant Laboratory which were confirmed tests.

BACTERIAL DIE-AWAY

Coliform bacteria are living organisms and as such are subject to their environmental conditions. Transferring them from the protected environment of the host to a more antagonistic environment, such as that of the Bay waters, will usually cause a substantial decay or die-away of the organisms. Although several investigators have looked into the mechanics of bacterial die-away in receiving waters, there still is no simple and sure method for estimating the concentration of organisms that will persist in receiving waters.

Bactericidal Action

In 1956 Orlob (5) summarized the literature that had been published. The major mechanism of mortality seemed to be that of bactericidal action or biological action in sea water. The general expression that was presented for mathematically describing the die-away characteristics was a first order equation established by Chick's Law (6) written as follows:

$$N_t = N_o 10^{-k(t-t_o)}$$

where N_t = the number of surviving organisms after time t

N_o = the initial bacterial population at t = 0

t_o = the lag time between the discharge of bacteria into the receiving water and the beginning of a definitive die-away

k = die-away constant.

This equation should be used only for relatively short periods because of a pronounced decline in mortality rate after about a 90 to 99 percent mortality.

Orlob pointed out that lag time and die-away constant will vary substantially with temperature and the presence of varying amounts of nutrient materials. Other minor bacteriological factors that may be present in the marine environment include predation by zooplankton, salinity, sunlight, pressure, bacteriophage, and heat-labile bactericidal

substances. For practical purposes it may be assumed that these factors operate universally in marine waters, hence may be combined into the one single factor of die-away constant.

It was also noted that bacterial mortality in sea water is susceptible to chlorination, pasteurization, and filtration, but its action is not completely destroyed by any one of these treatments. It would seem likely that the addition of chlorine would have the greatest effect on those organisms with the least resistance and would result in discharging the most resistant organisms. For this reason the assumption will be made in subsequent calculations that the die-away phase will be preceded by a lag time of 1.0 day when the effluent is chlorinated. The lag time for unchlorinated discharges will be assumed to equal 0.3 day, which is supported by the various results reported by the investigations and summarized by Orlob.

Die-away constants found by Orlob for incubation temperatures similar to those found in the East Bay waters and for the total coliform group ranged from 0.69 up to 1.59 per day. Gunnerson (7), in his study of existing outfalls in Santa Monica Bay, indicated a die-away constant of 1.35 per day. Vacarro (8) observed coefficients of mortality to be as high as 1.25 per day. On the basis of these references, the value of the die-away constant that will be used in this study to describe death rates due to bactericidal action alone will be 1.25 per day.

The only component that could be identified separately was predation by zooplankton. On the basis of work done by Ketchum, Ayers, and Vacarro (9) together with reported zooplankton concentrations in the central portion of San Francisco Bay of 8,530 per cubic meter, the K value for predation only was computed to be 0.13 per day.

Sedimentation

Orlob concluded that substantial die-away rates may be contributed to by sedimentation of organic materials with absorbed bacteria. He felt that this mechanism would not contribute to die-away beyond a limited zone of sedimentation. Gunnerson indicated that sedimentation is a major cause of coliform disappearance from surface waters. The considerable difference between coliform die-away rates in primary and secondary effluent suggests sedimentation as the cause, and this seems to be confirmed by later studies. Rittenberg (10) showed a progressive sinking of the zone of maximum coliform density and maximum turbidities as a sewage field moved away from an ocean outfall. However, Weiss (11) showed that bacteria previously absorbed on silt particles were found to be desorbed when placed in saline water, depending upon the type of silt, size of particles, and the chloride concentration. In comparing computed die-away rates to measured rates, Vacarro attributed the greater die-away rate in the measured results to sedimentation for which he had not accounted, based upon an accumulation of bacteria on the bottom silts. He also concluded that the apparent die-away from

sedimentation was more important than either dilution (in that case) or predation, but much less important than bactericidal action.

The die-away constant for reduction due to sedimentation can be estimated from the data presented by Vacarro to be 1.0 per day. However, Gunnerson in the Santa Monica Bay study found k values can be as high as 4.53 for primary effluent and 1.14 for secondary effluent. Because of the shallow water location of several of the discharges in the East Bay and the relatively high tidal currents in the vicinity of some of the discharges, it appeared that a die-away constant value similar to that for secondary effluent would be both conservative and more realistic. Therefore, the die-away constant value assigned to sedimentation for this study will be 1.15 per day. Combining the die-away constants for bactericidal action and for sedimentation results in an overall constant of 2.40 per day.

Dilution

The initial dilution of the effluent and subsequent dispersion of coliform organisms throughout the Bay is a third major factor in total coliform die-away. In this study, the dilution and dispersion was computed using the data from the Bay Model as described previously in this section.

BACKGROUND COLIFORM BACTERIA

Three sources of data were used for determining the background coliform concentrations in the East Bay waters: 1) the EBMUD Water Pollution Control Plant, 2) the 1963-64 University of California study (1), and 3) miscellaneous data from the Regional Board. Unfortunately, most of the available data were prepared prior to the time that EBMUD began chlorinating its effluent. Prior to January 1968 the EBMUD plant practiced prechlorination for odor control and some suppression of coliforms. Since that time, both prechlorination for odor control and postchlorination for bacterial reduction have been practiced.

Figure 45 is a probability plot of coliform concentrations in the vicinity of the EBMUD outfall. The upper line represents the results of coliform samples taken bimonthly during the 19[illegible] study. The sampling point was located approximately 4,000 feet north of the end of the outfall. The lower line represents the data collected by the EBMUD staff during the fiscal year 1968-69. Although more data were available, only two sampling points are represented in the lower line located approximately 1,000 feet north and south of the outfall, respectively.

These two plots show the effect of postchlorination in the vicinity of the outfall, because median coliform counts were reduced from an MPN of 9,46[illegible] per 100 ml down to an MPN of 400 per 100 ml. Looking at the lower plot, the MPN value for which 80 percent of the values are lower is slightly in excess of 1,700 per 100 ml. Therefore, the coliform counts

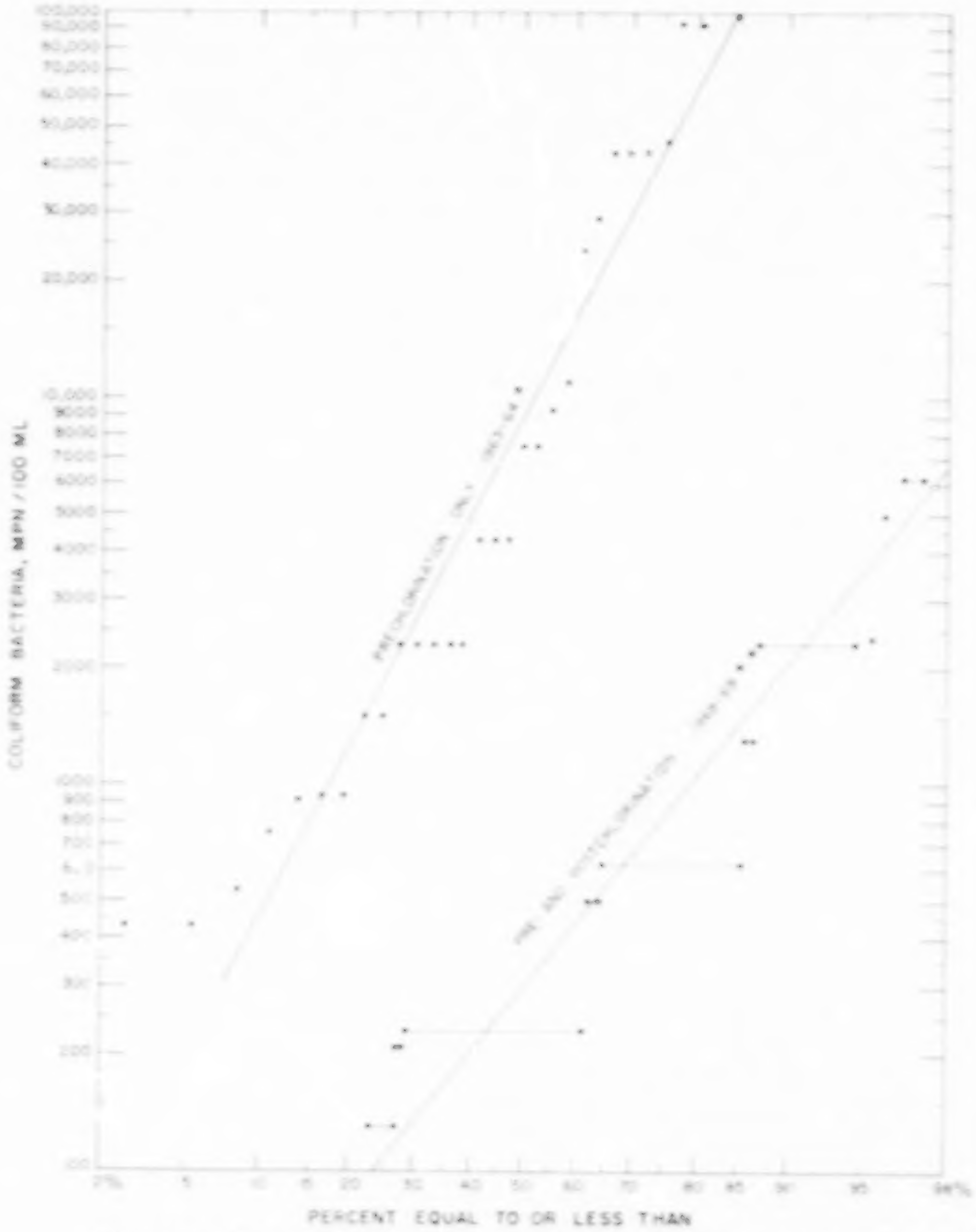

FIG. 45 PROBABILITY OF COLIFORM BACTERIA CONCENTRATION IN THE VICINITY OF THE EBMUD OUTFALL BEFORE AND AFTER POSTCHLORINATION (1963-64 AND 1968-69)

* [illegible]
** [illegible]

in the vicinity of the outfall do not always meet the Regional Board's objective for recreational waters.

It was noted in reviewing the available data that coliform counts in the immediate vicinity of the outfall were often lower than those at greater distances away. Conclusions are difficult to draw with reasonable confidence, but these data do illustrate the impact of chlorination on the receiving waters.

Figure 46 shows a generalized map of the central part of San Francisco Bay showing those areas that normally did not meet the coliform objective established for recreational waters before chlorination. This map, based upon data taken in 1964-65, was made available by the Regional Board and confirmed to a large degree by more recent background sampling done by EBMUD. It may be seen that coliform counts along the shoreline of Berkeley and Emeryville did not meet the coliform objective. However the near-shore waters north and south of the Berkeley Pier were normally within coliform limits. The waters along the south shore of Alameda as well as waters for some distance away from shore normally meet the coliform objective. Few data exist for waters within the Oakland Inner Harbor and San Leandro Bay. However, from limited data developed by EBMUD in 1965, it was concluded that these waters did not normally meet the water quality objective.

Figure 47 is a probability plot of data developed during the 1963-64 Bay study at a point located approximately one and three-fourths miles north of the Berkeley Pier and approximately three miles offshore. It was assumed that data collected at this point were representative of the entire area within that zone normally meeting the coliform objective. In this figure only about 70 percent of the samples fall within the MPN limits of 1,000 per 100 ml, indicating that the water quality objective is probably not met during only one or two months out of the year. Reviewing the available data, both fro… the 1963-64 studies and from EBMUD, the coliform counts are substantially greater for January and February than for other months. This may indicate that the reason occasional months do not meet the objective involves rainy season conditions, namely overflows and storm drainage.

In general, it can be seen that the only waters along the shoreline that presently meet the coliform objective for recreational waters are those along the south shore of Alameda. The near-shore waters adjacent to Alameda and Albany-Emeryville are normally within the objective. The additional loading due to untreated discharges from sanitary sewers and storm drains may be the reason for periodically exceeding the objective. All other waters do not normally meet the water quality objective.

ESTIMATES OF THE EFFECTS OF OVERFLOWS ON COLIFORM COUNTS

As indicated in the preceding paragraphs, the water quality objective for coliforms is presently being met in portions of the offshore waters

FIG. 46 AREAS OF SAN FRANCISCO BAY THAT NORMALLY DID NOT MEET THE SELECTED COLIFORM OBJECTIVES BEFORE CHLORINATION

SOURCE: BASIC DATA FROM REGIONAL WATER POLLUTION CONTROL BOARD, [illegible]

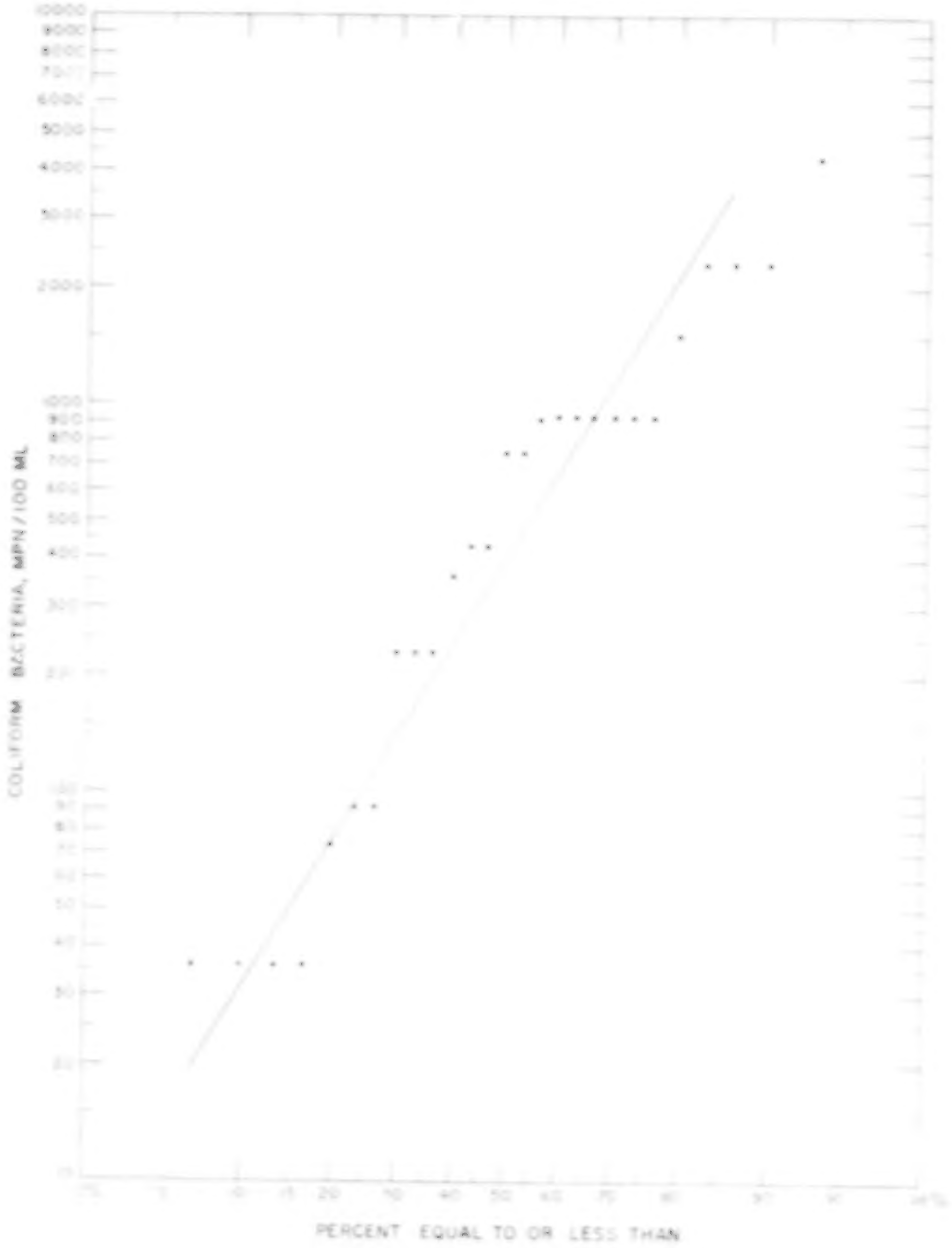

FIG. 47 PROBABILITY OF COLIFORM BACTERIA CONCENTRATION APPROXIMATELY 3 MILES OFF THE EMERYVILLE, BERKELEY, ALBANY SHORELINE, 1963-64

in the East Bay, except during one or two months of the year. These are believed to be the months during the peak of the rainy season, namely January and February. The question, then, is whether or not elimination of overflows would result in the offshore waters meeting the coliform objective during all 12 months of the year. Unfortunately, this question cannot be answered directly and with great assurance because of the many environmental factors involved.

The large quantities of storm drainage that occur simultaneously with overflows contribute large numbers of coliforms. Also, the persistence of coliforms during these periods of high turbidity and turbulence in the Bay cannot be predicted with assurance. The die-away rate of bacteria is subject to available organic material in the water needed for metabolism, together with the advective of large volumes of relatively fresh water into the near-shore Bay waters, possibly altering the normal bactericidal effects. However, using the normal die-away equation and die-away constants for bactericidal effects and sedimentation, together with the dispersion characteristics defined in the discussion on dissolved oxygen, estimates were made for the number of coliforms discharged to the receiving waters and the time necessary for the coliform counts to return to less than 10 per 100 ml above the normal background counts.

Because prechlorination and postchlorination are continued at the plant during periods when flow is bypassed around the sedimentation basins, and because the volume of water bypassed during a storm is normally not more than twice the average daily discharge, the overall increase in coliform discharge to the Bay during bypassing would be less than a factor of two. Therefore, in computing the coliform counts in the receiving waters resulting from overflows, the additional coliform discharged through the outfall will be disregarded.

Estimates for coliform concentrations discharged via overflows to receiving waters were based upon samples taken at Pump Station A and Pump Station E. This was done because samples at the EBMUD main plant were taken after prechlorination and therefore did not indicate actual coliform concentrations in the overflows. The coliform concentrations found for each zone were:

Zone 3: 3.7×10^8 MPN/100 ml

Zone 4: 1.5×10^7 MPN/100 ml.

Using the data developed in Section 5 for the average number and volume of overflows per year (11.5, producing an overflow of 67.5 MG in Zone 3 and 29 MG in Zone 4), the MPN of coliform discharged <u>per overflow</u> was estimated as:

Zone 3: MPN = 87.5×10^{16} coliform organisms per overflow

Zone 4: MPN = 15.9×10^{15}.

Distributing the coliforms in the volumes of near-shore waters, respectively, the average concentrations were found to be:

Zone 3: $N_o = 31.0 \times 10^3$ MPN per 100 ml

Zone 4: $N_o = 11.7 \times 10^3$ MPN per 100 ml.

The following equations were then used to estimate the time required before coliform counts returned to normal:

Zone 3: $N_t = N_o 10^{-2.4(t-0.3)}$, for values of t less than 5 days

Zone 4: $N_t = N_o 10^{-2.4(t-0.3)(2t)^{-1.1}}$

where N_t = coliform concentrations at time t, MPN/100 ml

N_o = coliform concentrations at time $t = 0$, MPN/100 ml

t = time, days

From these equations it was found that coliform counts contributed from overflows would be reduced to below an MPN of 10 per 100 ml in about 2.0 days in Zone 3 and 1.5 days in Zone 4. Based on the occurrence of 11 or 12 overflows per year, the East Bay waters would exceed the coliform objective for about 23 days per year (2 x 11.5).

It should be noted that the above computations were made for the average number of overflows per year and the average volume for each overflow. No accounting was made for extreme conditions, such as those that occurred during the 1968-69 rainy season. Obviously, the rain does not occur each year at equal intervals throughout the rainy season. A disproportionate number of storms will occur during the two months of January and February, resulting in greater numbers of overflows during those two months. It was therefore concluded that overflows contribute substantially to the high coliform counts normally found in the receiving waters during the months of January and February.

It would be difficult to conclude that elimination or disinfection of overflows would result in coliform counts lower than the water quality objective in the two near-shore areas, because storm sewer discharges also contribute large numbers of bacteria during the rainy season. However, the results of a bacteriological investigation of separate storm and sanitary sewer discharges by Burm and Vaughan (12) indicate that discharges from combined sewers (similar to sewage overflows) will contain about ten times more coliforms than discharges from storm sewers. Furthermore, the fecal coliform concentration in combined sewage will approach that of the total coliforms, whereas in storm flows the fecal coliforms seldom exceed 20 percent of the total coliforms.

Assuming that storm flow volumes from storm sewers in the East Bay Area exceed those from overflowing sanitary sewers by a ratio of five to one, the results of the preceding study suggest that overflows will contribute twice as many coliforms as will storm sewers. Almost ten times more fecal coliforms could be contributed by overflows than by storm sewer discharges. Also, overflows are known to contain fecal matter of human origin, whereas storm sewer discharges contain wastes that are primarily of non-human origin. Therefore, elimination or disinfection of sewage overflows is recommended, although storm sewer discharges would continue without disinfection.

SECTION 10

REVIEW OF POSSIBLE COMPONENT SOLUTIONS TO THE PROBLEMS OF OVERFLOWS AND BYPASSES

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SECTION 10

REVIEW OF POSSIBLE COMPONENT SOLUTIONS TO THE PROBLEMS OF OVERFLOWS AND BYPASSES

The problems resulting from storm water infiltration that have been described in this study may be summarized in the order of their importance as follows:

1. Pollution of San Francisco Bay caused by discharging floatable grease and large numbers of coliform bacteria via sanitary sewer overflows (intersystem and system) and water pollution control plant bypasses.

2. Operational problems at the water pollution control plant resulting from large quantities of grease, sand, and silt carried by extraneous flows. The deposition of sand and silt is the main reason for plant bypassing.

3. Property damage, public health hazards, and general nuisance caused by sewage overflowing (intrasystem overflows) onto streets from sewer manholes and appurtenances within the sewer system. These problems result from high flow rates that exceed the capacity of the sewers.

As the study progressed it became apparent that no single solution would resolve all of these problems. Therefore, a series of possible component solutions was developed from which combinations could be selected and further evaluated for use in alleviating the problems in the East Bay Area.

This section is devoted to a brief review of all of the possible component solutions derived from 1) an extensive literature review and 2) observations made and ideas developed during this investigation. Those component solutions that show promise for application to the problems in the East Bay Area are evaluated further in Section 11.

COMPONENT SOLUTIONS PRESENTED IN THE LITERATURE

The many approaches found in a review of the literature for solving the problems of excess wet weather flow fall into three general categories: 1) reduce infiltration to control overflows, 2) improve hydraulic design to control overflows, and 3) provide treatment for overflows.

REDUCE INFILTRATION TO CONTROL OVERFLOWS

Storm water infiltration may occur from many different sources, including:

1. Illicit connections.

   a. roof leaders
   b. basement drains (both pump and gravity)
   c. foundation and window well drains
   d. catchbasins and paved areas.

2. Submerged manhole covers.

3. Street catchbasins.

4. Groundwater or vadose water entering into broken or damaged sewers.

The two main solutions found for reducing or eliminating these sources were to 1) disconnect direct connections and 2) inspect and repair faulty sewers.

Disconnect Direct Connections

The first phase of a program to disconnect direct connections involves locating each direct connection, including illicit connections. This process usually requires an intensive type of survey, several of which were described in the literature. They ranged from house-to-house visual observations to the use of various tracers including marked objects, liquid dyes, and smoke testing. The results of these surveys depend largely upon the sincerity and initiative of the individuals involved.

The second phase of the program involves the actual disconnecting. Where private property is concerned, a frequently used approach is to submit a letter to the property owner that has been signed by the mayor, or other individual of similar authority, requesting the correction of the illicit connection. The letter normally describes the ordinance by which the request is made and stipulates a time limit during which the disconnection is to be accomplished. Results obtained from this kind of approach have been mixed. Some cities have had a reasonable degree of success, while others, lacking cooperation from their citizens, have had total failure.

Disconnections of direct connections found in public rights-of-way do not involve the political ramifications associated with disconnections on private property. There are many street drains or catchbasins connected to sanitary sewers, although a storm sewer is available. The disconnection of these catchbasins would substantially reduce the quantity of storm water entering the sanitary sewers.

From the experiences described in the literature it was concluded that a successful program for disconnecting illicit sewer connections involves:

1. Available funds and personnel.

2. Complete cooperation of the citizenry.

3. Persistency by the agency in charge of the sewer system.

From the experiences of several of the East Bay cities, it was concluded that complete cooperation of the citizenry would be difficult to obtain. Also, the topography is such that many houses were constructed with basements, garages, and patios situated at elevations below the level of the street, so it would be difficult to conform in every case to the present city plumbing codes. These codes require that roof drains be discharged onto splash blocks. Although these codes are sound, complete conformance would present difficulties because water would flow down the slope from one property onto the adjacent property, particularly in the hilly areas. This situation would not be satisfactory. Furthermore, as seen from the results of the mathematical flow routing model (Section 5), the contribution from direct connections was found to be less than that from infiltration through pipe walls and other similar means, i.e., percolation.

In view of these considerations, embarking upon a program of completely eliminating illicit or direct connections from private property would not be a practical solution, although a selective program would be of value. But a program should be undertaken to disconnect all catch-basins from sanitary sewers that are located in public rights-of-way and reconnect them to storm sewers.

Inspect and Repair Faulty Sewers

The procedures for sewer inspection and repair may be used on both new and existing sewer systems. In a new sewer system, inspection procedures may provide the basis for accepting or rejecting new construction work. In an existing sewer system, they may be used to determine the actual location of leakage and the extent to which repairs are required. Repairs may be necessary in either case, and their extent would be defined during the inspection. For purposes of this discussion, the use of sewer inspection and repair will be limited to existing sewer systems.

Sewer inspection techniques now include hydrostatic testing, smoke testing, low pressure air testing, photography, and closed circuit television. Each has advantages and disadvantages, and some are more applicable to existing sewer systems than others. The determining factor in the selection of the technique is whether or not it will provide the necessary information for making the sewer repairs. For instance, if the pavement surface must be cut and the trench opened to

expose the broken or damaged sewer, the location of the failure must be known within close limits. Simple hydrostatic testing or smoke testing procedures will not necessarily provide this kind of accuracy. Air testing or closed circuit television, however, can easily provide the detail required.

There are three generally accepted methods for repairing sewers: 1) excavate and replace, 2) internal sealing, and 3) external sealing. The method finally selected for use will depend upon the condition of the sewer as determined by the inspection or by the character of complaint that may have been registered for that reach of sewer. Complaints may result from street failures caused by soil leaching or from flooding caused by debris accumulation at broken, offset joints. The method selected will also depend on whether the repairs are made on a "crisis" basis or on a properly scheduled program of repairs.

In many portions of the East Bay Area, the sewer systems are less than 25 years old. Their expected service life should be at least another 30 years. Therefore, the initiation of a total replacement program for this category of sewer would not be logical. The City of Oakland has initiated a 60-year replacement program for its sanitary sewer lines, with the older, problem sewers scheduled for replacement first. Obviously, this type of program will not provide an immediate solution for infiltration problems, but it is the only ultimate approach to the overall problem. Such a program would not be economically feasible if it were accelerated to completion in a short period of time because of the tremendous cost involved. Therefore, the judicious use of an interim program of inspection and sewer sealing could provide a substantial reduction in infiltration in the very near future.

IMPROVE HYDRAULIC DESIGN TO CONTROL OVERFLOWS

Two general solutions were found in the literature for improving the hydraulic capacity of the sewer system and thereby controlling intersystem overflows: 1) sewer modifications and 2) holding facilities. The objective is either to increase the capacity of the sewer so that the entire combined flow can be contained, or to provide sufficient storage capacity to dampen the peak flow so that overflowing would not occur.

Sewer Modifications

These procedures include any special design, chemical addition, or modification that would substantially increase the capacity of the sewer system, thereby reducing the number and quantity of overflows. It excludes any form of treatment for the water. The sewer modifications considered were:

1. Reduce the friction factor by chemical addition.
2. Increase the sewer capacity at constrictions.
3. Construct relief sewers.

4. Provide means for flow regulation.
5. Make use of natural system storage.

The addition of chemicals to reduce the friction factor in a sewer to improve its capacity was recently investigated by The Western Company, Richardson, Texas. This alternative should be considered as a solution only for local problems. It would not seem practical to add chemicals simultaneously in all areas of the sewer system because of the physical limitations and the cost involved. Therefore, chemicals would be added only at locations where major constructions occur, or where replacement of a long reach of large sewer would otherwise be required. This alternative, then, is very similar to enlarging the sewer at locations where constrictions are present. However, the general concept of improving the sewer capacity by either reducing friction or by enlarging the pipe section to increase its capacity in a critical area is definitely a possible solution to local problems.

Construction of relief sewers may be necessary in those areas where the downstream capacity is inadequate to carry the combination of dry weather flow and extraneous flow. Such a sewer may be either parallel with the existing sewer or connected to a different sewer having a greater capacity. Again, this alternative would be applicable to local problems.

Flow regulation can be provided by installing either manually or automatically operated regulating devices in the sewer system. These devices help make use of the natural storage in the sewer system. Obviously, the best application of this alternative would be where sewers are large but do not flow full, except possibly at high flows which occur infrequently. This additional storage capacity could be utilized to reduce the peak of the discharge hydrograph. This procedure is presently being used to some degree in the East Bay system. During periods of high flow, as the capacity of the treatment facilities is approached, various actions are taken to reduce the flow into the plant. These may involve the closing of control gates or the stopping of pumping stations which cause portions of the interceptor and adjacent trunk sewers to fill to the overflow position. If flow to the plant cannot be resumed because of a continuation of high extraneous flow, then overflowing must take place.

Natural storage is the volume remaining within the sewer above that required for dry weather flow. Regulation, mentioned above, is simply the attempt to control and make full use of this natural storage. The presence of constrictions in a sewer system inherently makes use of the natural storage occurring upstream from that point. Of course, natural storage can be used only if it is available. Because the problems under consideration in this investigation involve an existing sewer system, the natural storage has been previously established and is fixed. Therefore, the available storage may or may not be in the most advantageous locations within the system. Some authorities contend that sewers should be initially overdesigned to provide additional storage

capacity, inferring that the additional capacity is available at a reasonably low cost. New sewer system design is not involved in this investigation; therefore, a complicated and extensive system of flow regulation, necessary to make use of the natural storage in the existing sewer system, would not be a practicable solution.

Each of the above solutions is applicable in reducing the number of overflows that occur locally within the sewer system. However, except for flow regulation, these sewer modifications do not present a general solution to the complex sewer system of the East Bay. Flow regulation may be applicable to a problem of this magnitude but would be technically difficult to apply in this case. Moreover, seven different jurisdictions are involved in the sewerage systems. Obtaining complete agreement by all parties as to the facilities needed, the operational requirements, and the financing would be difficult. Therefore, the applicability of flow regulation would also be questionable for the East Bay Area.

Holding Facilities

Holding facilities are devices designed to provide storage capacity necessary for dampening the hydrograph peak. They provide a substantial period of quiescence for the water, causing a natural separation of settleable and floatable material. Any treatment of combined flow obtained with these facilities would be incidental to the prime objective of dampening the peak flow rate. There are two types of holding facilities: on-line and off-line.

On-line facilities are those which receive both dry weather and wet weather flow. The advantage of an on-line holding basin is that normally the hydraulic gradient can be maintained so that pumping of either the overflow or the accumulated sludge would not be required. The disadvantage is that it must be constructed along the existing sewer and therefore land must be available. The land area requirement is substantial and this limits its applications to those locations where trunk sewers are constructed in relatively open areas.

Off-line holding facilities are those which do not receive flow during dry weather; in other words, they are used only during wet weather conditions. Off-line facilities have been used more commonly in the United States than any other type of overflow control. The greatest limitation to this alternative, as in the case of the on-line facilities, is the land area requirement. Off-line holding basins may receive overflow from the trunk sewer either by gravity or by pumping, and may be built either above grade or below grade, depending on the soil and geological conditions.

Thus, the application of holding facilities, either on-line or off-line, will depend upon the availability of adequate land area. It appears that land may be available in certain areas along the trunk sewers discharging into the north interceptor and in areas along the most south-

erly reach of the south intercepter. Although limited to these areas, holding basins definitely represent an alternative solution to the problem of overflows in the East Bay Area.

PROVIDE TREATMENT FOR OVERFLOWS

Two general methods were presented in the literature for treatment of overflows: 1) the use of flow separation devices and 2) the use of conventional treatment devices.

Flow Separation Devices

Several types of flow separation devices have been designed and built to remove pollutants from overflows. Most of these have been on-line units. The use of such devices in the United States has been relatively limited, but several have been under investigation in Great Britain. These devices include:

1. Low side-weir overflow
2. Stilling pond overflow
3. Vortex with central overflow
4. High side-weir overflow with storage
5. Spiral flow separator and overflow

The advantage of the low side-weir, stilling pond, high side-weir, and spiral flow separator overflow facilities is that pumping of either the through-flow or the overflow would not be required if they have been designed properly. Unless substantial elevation differentials are available or unless upstream sewers can be surcharged several feet, the vortex overflow may require pumping of either the overflow or the underflow. The low side-weir device was found to be less efficient than the other devices, both from the hydraulic and the solids removal standpoint. The results of solids removal studies reported for the stilling pond design varied substantially. Certain variations in design produced improvements whereas others apparently did not, and the differences were difficult to explain.

The vortex with a central overflow is not satisfactory unless it is designed with an appreciable storage capacity. The advantage of the vortex unit with storage capacity is that the first flush from a short-duration storm enters the storage chamber before overflowing begins. This means that the greatest quantity of polluting material is collected and held in the vortex unit. This is an attractive characteristic but one that can also be obtained with other devices.

The high side-weir overflow with storage capacity is also designed to capture the first flush before overflowing occurs. This is accomplished by limited storage capacity provided either in a section downstream from the overflow weirs or by increasing the height of the overflow weirs. The dry weather flow normally passes through the unit via an outlet at the downstream end of the storage chamber. During wet

weather flow, this outlet acts as an orifice, permitting a portion of the flow to continue along the sewer, forcing the remainder to overflow. The performance of this type of unit has been found to be better than the types discussed previously.

The spiral flow separator involves a channel with either a single or a double curve. The channel section is designed so that the main flow passes along the main channel and the overflow passes over the wall of the channel, but the curve in the sewer causes solid material to settle out in a quiescent zone produced along the inside of the bends. The settled material is continuously removed by the flushing action developed by water being drawn off through slots located in the settling zone. Because of the simplicity of the design and the results reported in the literature for this device, it is a possible alternative in those areas where partial treatment is necessary but large land areas are not available.

It should be noted that each of the preceding overflow devices was designed specifically for the removal of settleable materials. In this investigation, the problem of floatable materials is greater than the problem of settleable solids. Each device, however, can be fitted with scum boards or baffles that will tend to reduce the overflow of floatables. Unfortunately, the results reported from the little work done along this line were not encouraging. The question of most concern is whether the detention time is sufficient to permit the floatable materials to rise to the surface before being swept over the weirs. More experimental work is needed to fully answer this question. Also, if flow separation devices were used and the underflow sent to the treatment plant, the existing grit removal and handling facilities would require additional study and modification.

The available evidence does not warrant recommending the use of flow separation devices.

Conventional Treatment Devices

Very few of the conventional treatment methods have been applied to the treatment of overflows in the past because dilution was considered to be satisfactory. Another reason was the inherent cost in the treatment of large intermittent volumes of combined sewage. The following list of unit processes received attention as treatment methods for combined sewage overflows.

1. Screening
2. Filtering
3. Flotation
4. Sedimentation
5. Disinfection

6. Ponds
    a. aerated lagoons
    b. stabilization ponds
7. Flow through biological systems.

Physical conditions, such as availability of land in the East Bay Area, limit further consideration to only four of these processes: screening, flotation, sedimentation, and disinfection.

Fine screening, flotation, and sedimentation processes are being evaluated for other locations in the United States, under the sponsorship of the FWQA. In most cases these evaluations are being made on combined sewer systems that have waste discharge characteristics somewhat similar to those found in this study. Each of these methods is applicable to removal of grease and floatable materials and should be considered as a possible solution to problems in the East Bay Area. However, past experiences with fine screening at the EBMUD Water Pollution Control Plant indicated a problem of severe blinding because of grease. Improvements in design and recent experience at other installations have indicated that by proper design and selection of equipment, screening can be considered as an applicable process. Flotation processes have the added requirement of process control, including recycle rates, recycle pressures, etc.

Disinfection of overflows is necessary to reduce the coliform counts so that the receiving waters are maintained within the water quality objectives or, at least, so that overflows are not the cause of the objectives not being met. This, of course, would not account for storm sewer discharges that also carry large quantities of coliform bacteria.

ADDITIONAL COMPONENT SOLUTIONS

In addition to the component solutions described in the literature, three other approaches were developed for application to the East Bay Area conditions: 1) revisions and additions to the water pollution control plant, 2) improved sewer design and construction practices, and 3) application of multipurpose concepts.

REVISIONS AND ADDITIONS TO THE WATER POLLUTION CONTROL PLANT

None of the component solutions presented in the literature review involving treatment of the overflows would reduce the quantity of settleable material discharged to the water pollution control plant. Each of those alternatives simply helps to reduce the pollutional characteristics of the water discharged to the Bay, but in so doing would discharge most of those pollutants to the existing treatment plant. As discussed earlier in this report, one of the problems of the existing plant during high extraneous flow is that of large quantities of sand and silt settling in the sedimentation tanks. The sludge is deposited

in such large quantities and at such high rates that the sludge collection equipment cannot cope with it and must be stopped before failure occurs. To avoid mechanical failure, the flow heretofore has been bypassed around the sedimentation tanks when the combined flow reaches approximately 120 mgd.

As indicated in Section 6, prevention of bypassing at the water pollution control plant would prevent an annual discharge of 780 MG of an essentially untreated combined sewage. This volume amounts to about 43 percent of all the combined sewage and extraneous flow discharged by overflows and plant bypassing annually. However, this percentage is not directly indicative of the relative effect of bypasses on the pollutional problem, because of the dilution and dispersion produced by discharging through the outfall to deep water. Correction of the single problem of grit removal and handling would almost completely eliminate bypassing, thereby ensuring primary treatment of a major portion of the extraneous flows before it is discharged to the Bay.

If the settleable material, previously discharged to the Bay via overflows, is to be discharged to the plant, the total quantity of settleable material will be substantially increased during periods of overflow. If no additions or corrections were made at the plant, it would follow that bypassing would be necessary at even lower rates than before.

IMPROVED SEWER DESIGN AND CONSTRUCTION PRACTICES

A summary of suggested sewer design and construction procedures that would contribute to reducing infiltration is presented in Table 25. Many of the recommendations are applicable only to new sewer design and construction, hence apply to projects in the East Bay Area involving extensive sewer replacement.

In addition to design practices, two problem areas were discussed in which construction procedures should be reviewed with the aim of reducing infiltration: 1) inspection, and 2) administration. The inspection problem involves a shortage of competent inspectors. During the peak construction season, the Public Works Department often cannot provide the number of inspectors necessary; therefore, inexperienced men may be used who may sincerely conclude that certain work is satisfactory although it does not completely meet the specifications. For this reason, inexperienced personnel should be provided with sufficient training to give them a better understanding of the requirements of their position.

Administration problems involve contract administration and departmental jurisdiction. Contract administration difficulties arise if a nontechnical governing board refuses to support the recommendations of the technical staff. In such cases the bargaining position of the technical staff is seriously impaired. Departmental jurisdiction problems center around the question of which department is in charge of

TABLE 26

SUMMARY OF SUGGESTED IMPROVEMENTS IN SEWER DESIGN AND CONSTRUCTION PROCEDURES

Problem	Recommendation
Soils investigation	
• Laying pipe on unstable soil, permitting excessive movement of the pipe during compaction or in subsequent long-term settlement. • Inability to compact pipe bedding because of "[illegible] action." • Unrealistically high compaction requirements, causing excessive compactive efforts and possible damage to pipe.	• The subsurface investigation must be sufficiently comprehensive to provide the design engineer with the necessary information to prepare a safe, workable, and economical design. The investigation should include information on: 1. The presence and location of rock. 2. The elevation of groundwater. 3. The nature of the soil relating to: a. compactive characteristics b. stability of trench walls c. bearing strength of subsoil.
Selection of pipe materials	
• Inability of pipe to withstand overburden of trench backfill and/or live loads. • Inability of pipe to resist chemical attack.	• Physical and chemical characteristics of pipe materials should be compatible with the physical, chemical, hydraulic, and economic requirements of the system. • Consider joint design availability when selecting pipe material. • Consider bedding and backfill materials when selecting pipe materials and strength classifications.
Selection of pipe jointing materials	
• [illegible], root intrusion, and [illegible] joint leakage. • Failure of joints or flexibility is not available.	• The joint should be standard, with the pipe material specified; be simple, requiring very little skill and time to make; and give a positive indication when it is made and properly seated. (For example, a telltale sound that indicates positive seating or a mark that identifies proper insertion). • The completed joint should be tight and should offer reasonable flexibility but must be capable of transmitting shear loads to prevent offsetting at joints.

TABLE 24 (concluded)

Problem	Recommendation
[illegible]	
• [illegible]	• [illegible] AASHO No. 57. Do not use sand or smooth surface gravel.
[illegible]	
[illegible]	
• [illegible]	• [illegible]
• [illegible]	• [illegible]
• [illegible]	• [illegible]
[illegible]	• [illegible]
• [illegible]	• [illegible]
• [illegible]	• [illegible]*
[illegible]	
• [illegible]	• [illegible] to permit control of the project.

* [illegible] Federal Water Quality Administration.

controlling the connecting of house laterals to sewer mains. Neglecting to define this area of jurisdiction promotes uncertainty between the departments and may result in little if any control over the type or quality of connections being made.

Miscellaneous construction procedures involve the correction of house connections during the replacement or repair of main sewers. In addition to replacing the house connection, a new alternative involving a product called Cychem[6], manufactured by the American Cyanamid Company, may be available. Cychem is a new plastic pipe which may be drawn into a house lateral or sewer in a deflated condition and then inflated and cured in place. The lining would be a tough, glass-fiber reinforced polyester pipe and would be continuous from the building out to the main sewer. This pipe could also be used to line the main sewer, forming a single continuous piping system. Because of its continuous nature it would minimize leakage and root intrusion. The simplicity of installation and the result may outweigh the legal problems of maintenance responsibility; hence this alternative should not be overlooked.

APPLICATION OF MULTIPURPOSE CONCEPTS

The areas of the East Bay that offer sufficient land for the construction of holding basins are located along the waterfront and in publicly held land, such as neighborhood parks. Except for the Albany area, the land use along the waterfront zone is mainly industrial, heavy commercial, and residential. The residential areas generally consist of older dwelling units of stereotype construction. Most of the residents are in lower-middle and lower income groups, and a high percentage are in minority groups. If sufficient land area must be placed under public jurisdiction for the purpose of constructing storm water holding tanks, it would be desirable to incorporate aesthetically appealing landscaping and possibly recreational facilities, such as playgrounds or tennis courts. The added cost of landscaping or recreational facilities would not be high from the standpoint of the project, but would substantially increase the social acceptance and social value of the project. This concept would, of course, be applicable only for holding tanks that could be placed underground and that require sufficient land area to make the site desirable for recreational activities. Also, the site would have to be in or near a residential neighborhood. Any such facilities designed and constructed within an industrial or commercial area would probably be suitable only for such uses as public parking.

For either or both of the above reasons, it is desirable that better arrangements for grit removal and handling be considered for the existing water pollution control plant.

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[6]Mention of commercial products does not imply endorsement by the Federal Water Quality Administration.

CONCLUSIONS

On the basis of the foregoing discussion, the following list of component solutions could be effectively used to solve problems of overflows and bypasses in the East Bay Area:

1. Improve the sewer system.

   a. Complete the sewer separation program immediately.
   b. Initiate a comprehensive program of sewer repair and replacement.
   c. Initiate a comprehensive program of sewer inspection and sewer repair by sealing to reduce infiltration and subsequent overflows.
   d. Institute a program for locating and disconnecting all street drains from the sanitary sewers.
   e. Initiate a program for correcting local problems produced by overflowing sewage, both through manhole covers and intersystem overflows.

2. Provide new holding basins or overflow treatment facilities.

   a. Provide either on-line or off-line holding facilities to improve the hydraulic capacity of the sewer system and thereby control overflows.
   b. Provide treatment of the remaining overflows using one or more of the following facilities together with disinfection; 1) flotation, 2) sedimentation, and 3) screening.

3. Provide additions and revisions at the water pollution control plant to eliminate the need for bypassing the sedimentation facilities during wet weather flow.

4. Where needed, improve sewer design and construction practices for extensive sewer replacement programs.

5. Consider application of multipurpose concepts for design of holding or overflow treating facilities.

SECTION 11

EVALUATION AND SELECTION OF ALTERNATIVE SOLUTIONS

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SECTION 11

EVALUATION AND SELECTION OF ALTERNATIVE SOLUTIONS

On the basis of the discussion in Section 10, the following categories of component solutions will be further developed in this section, and cost estimates will be presented so that comparisons can be made.

1. Improvements to the sewer system.

2. New holding basins or overflow treatment facilities.

3. Improvements to the water pollution control plant.

This evaluation will be followed by the selection of the most feasible combination of component solutions and the development of a schedule for implementing the recommended plan.

EVALUATION OF COMPONENT SOLUTIONS

IMPROVEMENTS TO THE SEWER SYSTEM

The estimated length of sanitary sewers in the study area is approximately 1,700 miles. This estimate was based on discussions with the engineering departments of each of the six East Bay cities, and does not include the interceptor sewer which is approximately 21 miles long. In addition, the total length of house connections or building sewers was estimated at 1,500 miles, based on an average length of 50 feet per building and 156,600 buildings. As discussed previously, an estimated 4 percent of the area is presently served by combined sewers. Therefore, approximately 68 miles of combined sewers still exist in the study area, and most of these lie within the commercial-industrial portion of the city of Oakland. The remainder are located in isolated areas throughout most of the study area.

The estimated construction cost for each of the component sewer revisions is shown in Table 26. No estimate was included for repairing or replacing house connection sewers. Even if all of the work outlined in Table 26 were accomplished, there would still be substantial contributions of infiltration from roof leaders and house connections. However, based on conclusions drawn in Section 6, the reduction in flow provided by the sewer revisions would probably permit the sewers to handle the increase in flow during wet weather.

The benefits of repairing all house connections to eliminate gross infiltration completely may not presently be worth the cost, when it is considered that the component sewer revisions are expected to eliminate

TABLE 56

ESTIMATED CONSTRUCTION COST OF COMPONENT SEWER REVISIONS*

1. Complete sewer separation program	
a. 12-in. diameter sewer, 68 miles @ $32.00 per lf, including repaving	$11,500,000
b. Manholes @ 300 feet apart, 1,200 @ $750	900,000
c. Reconnect house connections, 12,000 @ $250	3,000,000
Subtotal	$15,400,000
Contingencies(25%±)	3,850,000
TOTAL	$19,250,000
2. Repair and seal remainder of sewer system	
a. Replace about 15 feet per 300 feet of sewer, 430,000 lf @ $32.00	$13,750,000
b. Clean, inspect, test, and pressure grout seal (hilly areas only) 2,870,000 lf @ $3.00/lf**	8,600,000
c. Clean, inspect, test, and pressure grout seal (flat areas only) 5,740,000 lf @ $4.30/lf***	24,700,000
Subtotal	$47,050,000
Contingencies (25%±)	11,750,000
TOTAL	$58,800,000
3. Locate and disconnect catchbasins	
a. 500 catchbasins @ $650	$ 315,000
Contingencies (25%±)	85,000
TOTAL	$ 400,000
4. Replace selected sections of sewer to eliminate bottlenecks. Note: Estimate based on extrapolation of detailed estimate made for Drainage Area No. 29.	
Sewer improvements for Drainage Area No. 29	$ 215,000
Sewer improvements for remainder of study area (42,000-1,000)/1,000 x $215,000	$ 8,800,000
Subtotal	9,015,000
Contingencies(25%±)	2,255,000
TOTAL	$11,270,000

*Estimating prices based on late 1972 values (ENR Index [illegible].F. = 1900).
**Based on an average sewer size of 8-in. diameter.
***Based on an average sewer size of 12-in. diameter.

enough flow to alleviate the problem. The estimated cost for repairing house connections by replacement with new pipe, using a cost per connection of $500, was about $78,300,000 (156,600 x $500).

Complete Sewer Separation Program

The cost estimate for completing the sewer separation program that was initiated as a result of the "1941 Sewage Report" was based on an average pipe diameter of 12 inches. Presently the combined sewers serving these areas are larger than 12 inches. However, with the separation of the two flows, the sanitary sewer should be smaller than the combined sewer. Also, the sewers in the commercial-industrial area may range in size up to 18 inches, but the sewer revisions in the isolated areas would likely require a size of eight inches. Manhole spacing was assumed to be an average of 300 feet. Because the new sewer is proposed to be the sanitary sewer, the present house connections would have to be either shortened or lengthened and connected to the new sanitary sewer. A contingency factor of 25 percent was applied to each estimate. The total cost estimate for completing the sewer separation program was $19,250,000 based on construction costs in 1972 (ENR Index S.F. = 1900), the approximate mid-point of construction if a program were started immediately. This estimate does not include administration and engineering costs, which will be applied when the combination of components are compared, nor does it include a cost for traffic and business interference, as is normally the case.

Repair and Seal Remainder of Sewer System

The estimate for repairing and sealing the remainder of the sewer system (96 percent) was based on the pressure grout sealing method. The average sewer size was assumed to be eight inches; in the flatter areas, it was assumed to be 12 inches. All of the sewers involved in sealing would be cleaned and inspected. Because the sewers in most of the areas are quite old, it was assumed that several joints of pipe between each manhole reach would require replacing before sealing could be effective. This amount was assumed to be 15 feet for every 300 feet of sewer. It should be noted that the 15 feet of sewer would not necessarily be consecutive sections, nor would each manhole reach require sewer replacement. The cost for sealing was based on estimating prices from a local sealing contractor. The total estimated cost for repairing and sealing the remainder of the sewer system was $58,800,000.

Locate and Disconnect Catchbasins

The estimated cost for locating and disconnecting catchbasins that are presently connected to sanitary sewers was intended to include only those within a reasonable distance from a storm sewer. Catchbasins that are presently located within the combined sewer areas are not included in these figures, because it was assumed that the storm sewer would be the "existing sewer" after construction of new sanitary sewers; hence the catchbasins would already be connected to the proper sewer.

The number of catchbasins to be disconnected was estimated on the basis of counts made in a given area and then projected over the study area. The accuracy of the number is questionable but was believed to be indicative of actual conditions. The total estimated cost for disconnecting these catchbasins was $400,000.

Replace Selected Sections of Sewer to Eliminate Bottlenecks

Replacement of selected sections of the sewer system in order to eliminate bottlenecks would first involve a careful investigation of each drainage area to locate the problems. Such a study was made in Drainage Area No. 29 as part of this investigation. In that area several reaches of sewer required enlarging so that the peak flow could pass through the pipe without surcharging and overflowing. These revisions were estimated to cost $215,000. It was then assumed that similar revisions would be necessary throughout the study area in conjunction with the use of overflow treating facilities. Therefore, the total estimated cost for replacing selected sections of sewer (usually replaced with larger sizes) was derived by prorating on the basis of relative areas; hence the total estimated cost was $11,270,000.

NEW HOLDING BASINS AND OVERFLOW TREATMENT FACILITIES

New Holding Basins

Holding basins are limited in application as component solutions to infiltration problems because they can only act to dampen the peak flow rate, thereby helping to prevent sewer overflows. During a storm of an abnormally long duration, a holding basin would be of little value. Fortunately, there is sufficient time between storms to permit rising and falling flow rates to dampen.

A holding basin installation was investigated for application to Drainage Area No. 29. The selection of the holding basin dimensions was made by using the flow routing model (described in Sections 5 and 6) to compute an inflow hydrograph, followed by using a storage model for computing the holding basin effect on the hydrograph. The resulting outflow hydrograph was then reintroduced into the flow routing model and routed through the remainder of the system. A series of trial runs was made in order to find the dimensions needed to eliminate sewer system overflows.

Certain sewer modifications were found to be necessary, either upstream or downstream from the holding basin, to make the connections to the holding basin. Therefore, the construction cost estimates for holding basins include the estimated cost for the necessary sewer connections or revisions.

The location of the holding basin in Drainage Area No. 29 was selected because the peak flow was exceeding the sewer capacity in that area and because land was available for installation of the facility. A

local park was assumed to be available for this purpose. If holding basins are used, a study of available land would be made in other drainage areas throughout the East Bay Area. The best way to determine the required sewer revisions and the proper size of holding basin would be by use of the computerized flow routing model developed during this investigation and used for Drainage Area No. 29.

Two different designs of holding basins were considered for application to Drainage Area No. 29. Each design included a fixed roof. Such a roof could then be incorporated into the existing recreational facilities of the park, e.g., tennis or basketball courts. The alternative designs consisted of:

1. Rectangular units with mechanical cleaning equipment.

2. Self-cleaning arrangements using sewage for flushing the settled solids.

The design involving mechanical cleaning equipment may take several different configurations. However, based on estimated mechanical equipment capital and operating costs, the different basin designs are essentially equal. Therefore, the final selection would depend upon structural design considerations and characteristics of the specific items of mechanical equipment. Because the roof span would be greater for square tanks than for rectangular tanks, the latter would be preferred. Also, the use of a low-head moving bridge type of cleaning equipment would be preferable because less maintenance would probably be required during the summer season. For these reasons, the rectangular shaped holding basins with moving bridge cleaning equipment are recommended for consideration as a component solution.

The self-cleaning design for a holding basin would be arranged to operate without mechanical cleaning equipment. The basin would consist of one or two parallel channels constructed with bottom slopes of not less than 5 percent beginning with an Ogee spillway at the head-end of the channel. This spillway would be arranged to assist the hydraulic flushing of the solids. Initial and final flushing action would be derived from the sewage flow itself. The solids would be flushed into a pumping sump at the back end of the channel. The pumps would meter the water back into the main sewer as well as return the solids to the sewer to continue on to the plant. The disadvantage of this design is that a substantial hydraulic gradient is necessary to provide the flushing action needed to wash the settled sludge to the pumping sump, resulting in generally deep construction.

The inlet to the basin would be so arranged that water would enter only after the level in the sewer reached a predetermined level. This could be accomplished by side channel weirs set, for example, at the normal depth for peak dry weather flow or by a more positive flow control device.

The estimated cost for a typical holding basin is shown in Table 27. It was concluded that the two designs discussed above would have about the same construction cost, although the self-cleaning arrangement would be slightly lower. The cost estimate includes the complementary sewer improvement. The total cost for providing holding basins in strategic locations throughout the study area was estimated by extrapolating the estimated cost for Drainage Area No. 29 by the ratio of the two areas. Therefore, the total cost for holding basins in the East Bay Area would be $19,700,000 ($470,000 x 42,000 acres/1,000 acres).

TABLE 27

ESTIMATED CONSTRUCTION COST OF TYPICAL HOLDING BASIN*

Note: Design based on Drainage Area No. 29 serving about 1,000 acres.

Structural concrete and earthwork		$165,000
Mechanical		60,000
Electrical		25,000
Site improvements		45,000
Sewer revisions		80,000
	Subtotal	$375,000
	Contingencies (25%)±	95,000
	TOTAL	$470,000

*Estimating prices based on 1972 costs (ENR Index S.F. = 1900).

Overflow Treatment Facilities

Of the several methods reviewed for the treatment of overflows, none seemed more suitable for removing floatables than either dissolved air flotation, fine screening, or plain sedimentation, each in combination with disinfection. The preliminary design parameters used for each of these alternatives are included in Appendix Table IV-1.

Dissolved Air Flotation. Because the dissolved air flotation process is known for its effectiveness in removing floatable materials, its application to overflow problems seems natural. However, the necessary equipment causes the cost of this alternative to be quite high. Also, no experience is available for the application of this process to the treatment of overflows such as those of the East Bay Area. Equipment is now being installed in San Francisco for the treatment of combined sewer overflows and data should soon be available on this application.

A cost estimate for the installation of an air flotation system was made for a typical overflow site. The results are tabulated in Item 1 of Table 28.

TABLE 58

ESTIMATED CONSTRUCTION COST OF TYPICAL OVERFLOW TREATMENT FACILITIES*

Note: Designs based on an average flow rate of 23 mgd and maximum flow rate of 35 mgd.

1. Dissolved air flotation	
a. Structural concrete and earthwork	$ 380,000
b. Mechanical equipment	280,000
c. Electrical	125,000
d. Influent and effluent sewers	250,000
e. Site improvements	100,000
f. Miscellaneous piping	65,000
g. Land	60,000
Subtotal	$1,260,000
Contingencies (25%) ±	320,000
TOTAL	$1,580,000
2. Fine screens (traveling water screens)	
a. Structural concrete and earthwork	$ 250,000
b. Mechanical equipment	280,000
c. Electrical	100,000
d. Influent and effluent sewers	250,000
e. Site improvements	125,000
f. Land	50,000
Subtotal	$1,055,000
Contingencies (25%) ±	265,000
TOTAL	$1,320,000
3. Plain sedimentation	
a. Structural concrete and earthwork	$ 430,000
b. Mechanical equipment	190,000
c. Electrical	90,000
d. Influent and effluent sewers	250,000
e. Site improvements	115,000
f. Miscellaneous piping	75,000
g. Land	65,000
Subtotal	$1,215,000
Contingencies (25%) ±	305,000
TOTAL	$1,520,000

*Estimating prices based on 1972 costs (ENR Index S.F. = 1900).

The air flotation installation was sized on the basis of an average flow rate of 23 mgd and a maximum flow rate of 35 mgd. The rate is based on 1,040 MG of overflow per year and 11.5 occurrences per year, each over a period of 9.5 hours. A total of ten sites would be required if no sewer separation is accomplished, nine if about half of the remaining program is completed, and seven if total sewer separation is accomplished. The surface area was selected for a 3-gpm per square foot overflow rate, and the recycle pump and pressurization equipment were sized for a 33 percent recycle rate.

The chlorination equipment estimate was based on the use of a sodium hypochlorite solution with variable discharge feed pumps which would diffuse the chlorine solution at the head-end of the flotation unit. The hypochlorite holding tank was estimated as a glass-fiber reinforced polyester tank sized so that it would contain enough chlorine solution for five average overflows. Feed rates were based on applying 25 mg/l of available chlorine to each overflow.

The air flotation unit would be preceded by a one-inch opening, mechanically cleaned bar screen and sewage pumps to lift the raw sewage into the treatment facilities. Special arrangements would be required in most instances to insure the prevention of reverse flow through the units caused by tidal variations, i.e., tide gates. Because each of these units would be constructed in a more or less densely populated area, the land area was estimated as small as possible to contain the unit. Assuming that public land, either in part or in whole, would be available in some instances, the average land cost was estimated to be $40,000 per acre.

This installation would be located near the waterfront where foundation conditions are very poor and where space for construction is limited. Therefore, adjustments were made in the cost estimates to account for groundwater, shoring of the excavation, and pile foundations. Also, in some locations an outfall to the Bay might be necessary.

The appropriate location for overflow treatment facilities would be in the vicinity of the present overflows or diversion structures and in certain locations where large trunk sewers meet the interceptor, even though an overflow structure does not now exist.

Fine Screens. Several types of fine screens are available that could be applicable to the treatment of sanitary sewer overflows. These vary from revolving drum and vibrating units to vacuum units with disposable filter media. A broad spectrum of screen sizes is available together with different materials of construction. In general, cost increases as the screen size decreases. All of these factors make selection of a specific screen at this phase of the investigation difficult. Therefore, cost estimates are presented here as an aid in evaluating the applicability of alternative component solutions, using the screen that appears to be most acceptable.

Two types of screens offered promise for the treatment of overflows: a traveling water screen and a fixed, self-cleaning screen. The traveling water screen would require the use of steam sprays to maintain the mesh free from grease build-up. It is also possible that steam sprays might be needed on the fixed screens. The advantage of the traveling water screen is that a lower pumping head would be satisfactory, whereas the entire flow would have to be pumped up and uniformly distributed over the fixed screen. Neither of these screens has been used in this service, but their features make them promising enough to be considered for additional study.

Item 2 of Table 28 presents the cost estimate for the most promising screening alternative--traveling water screens. On the basis of first cost, this installation would be more attractive than dissolved air flotation. However, before any final design is attempted, a pilot study should be conducted to define the acceptability of screening and the design parameters.

The screening installation estimate was based on a mechanically cleaned bar screen followed by two parallel traveling water screens. Both screens would be fitted with steam sprays as necessary to keep the screen free from grease. All of the screens would be located inside a building. Chlorination was based on a system similar to that described previously for air flotation, except that covered chlorine contact tanks were necessary. In some cases, part or all of the necessary contact time may be obtained in the outfall sewer, between the screen installation and the point of discharge into the Bay. Therefore, the tank was sized to provide ten minutes of holding at peak flows, assuming the remaining five minutes would be obtained in the outfall sewer.

Plain Sedimentation. Plain sedimentation has the advantage of simplicity. However, simplicity does not mean that all mechanical equipment can be eliminated because the remaining sludge and scum in the tank must be removed. The sedimentation basin was sized for 30 minutes of holding, providing time for substantial fractions of the floatable material to rise to the surface. This holding time should be evaluated for desirable floatable removals before any final design is undertaken. The same number of installations would be necessary as for screening and air flotation. Also, their locations would be about the same, except for necessary changes to obtain more land area. Chlorination and pumping equipment would be similar to that for air flotation.

Item 3 of Table 28 presents the estimated cost for this alternative component solution. The cost estimates for plain sedimentation and air flotation were almost equal.

Disinfection. Each of the preceding alternatives for removing floatables was estimated including chlorination for disinfection. As indicated, the chlorine would be in the form of sodium hypochlorite solution. Liquid or gaseous chlorine was considered to be a potential public safety hazard and therefore a last choice.

An estimated cost of chlorine per treatment installation would be about $2,700 per season, based on a 25 mg/L dosage of available chlorine, 23-mgd design flows for 9.5 hours at each event, 11.5 events per year, and $0.14 per pound of available chlorine. Although hypochlorite is safer than other forms of chlorine, there is a time limit beyond which it substantially loses its strength. Therefore, some planning and handling problems are inherent in its use.

An alternative to the purchase of hypochlorite solution is to generate an equivalent solution on-site as needed. Although no precedent has been established, an electrolytic treatment device is presently being evaluated for marketing that is claimed to make this alternative possible. The equipment is manufactured by the Pacific Engineering and Production Co. of Nevada.[7]

Probably the most ideal application of this type of equipment exists along the East Bay shoreline because of the availability of high chloride content Bay water. The water could be pumped from the Bay, through the electrolytic unit, changing some of the chlorine in the salt to the hypochlorite state. The solution would then be added to the screened or otherwise treated waste flow exactly as the sodium hypochlorite solution discussed previously. Although this technique is new, it should be studied as an alternative to purchasing and handling other forms of chlorine. A pilot plant could be established simultaneously with any other studies that might be made on overflow treatment facilities.

Conclusion. The construction estimates for the three different types of overflow treatment facilities in Table 28 are for single units. It was estimated that the equivalent of nine such units would be necessary to serve the entire study area if only 50 percent of the remaining combined sewers were separated. However, if all sewers were separated the required number would be reduced to seven. Therefore, on the basis of using fine screening, the total estimated cost for providing treatment facilities for the overflows in the total study area would be:

1. Before completing sewer separation, 9 @ $1,320,000 = $11,900,000.

2. After completing sewer separation, 7 @ $1,320,000 = $9,200,000.

It should be noted that the number of units selected here is presented as a means for estimating the cost of this alternative component solution. In the final analysis, it may be concluded that only two or three units would be desirable. In that case the flow rates would be adjusted accordingly, and some savings may be realized because of the increase in scale.

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[7]Mention of commercial products does not imply endorsement by Federal Water Quality Administration.

The fine screening alternative offers the lowest capital cost, the most reliable and positive results, and the smallest overall site. Therefore, fine screening is recommended as the most feasible means of treating overflows. However, pilot studies should be conducted prior to final design in order to better define the design parameters and operational problems involved in fine screening and to determine the effluent quality. If very strict standards are established on oil and grease in the effluent or in the receiving waters, then it is conceivable that the cost and operating problems of such fine screens may dictate the use of dissolved air flotation or some modification thereof.

IMPROVEMENTS TO THE WATER POLLUTION CONTROL PLANT

As discussed previously, the main reason that wet weather conditions present problems at the water pollution control plant is the influx of large quantities of sand and silt that settle in the sedimentation basins and damage the equipment. A secondary problem is that these large quantities of sand and silt are pumped into the digesters, thereby reducing the digester capacity. The present means of alleviating this problem is to bypass the flow during wet weather when the flow rate reaches approximately 120 mgd.

The continual upgrading of water quality standards may require conversion of the water pollution control plant from its present status as a primary plant to that of a secondary plant in the very near future. Therefore, any additions or modifications to the treatment plant proposed as solutions to infiltration problems should take this into account. A detailed engineering investigation has not been made of the existing facilities to date, so there is no sure way of knowing what type of treatment may be required or used in the future. However, a majority of the secondary treatment plants presently being planned or constructed use some variation of the activated sludge process. Therefore, that process was assumed applicable in this case for purposes of evaluating the alternative revisions to the plant.

Three basic alternative revisions were considered:

1. Modify the existing grit chambers.

2. Add new grit removal facilities.

3. Modify sedimentation basins and add sludge handling equipment to accept large quantities of sand and silt.

A summary of the design parameters used for sizing the various units involved in the revisions is included in Appendix Table IV-1.

Lengthen Existing Grit Chambers

The present grit chambers consist of five channels approximately 60 feet long and 12.5 feet wide, having the chain and bucket type of grit

removal equipment. The discharge from the channel takes place over a 12.5-foot long rectangular weir. Each channel was designed to receive the discharge from a constant speed pump. The only exception is that two of the pumps have two-speed motors, but the variation in discharge is only about 10 to 12 percent. Because of this design there is no significant change in flow rate between dry weather and wet weather conditions. The reduced grit removal during wet weather conditions seems to be caused by an increase in particle sizes smaller than the chambers were sized to remove. Although the chambers can separate substantial quantities of grit, the grit removal equipment is so placed that only part of the grit can be removed unless the chamber is taken out of service. As the chain and bucket conveyors reach the head-end of the channel, they must rise vertically through the incoming stream of flow immediately after the discharge from the pump. This causes a large part of the grit contained in the bucket to be washed out and returned to the bottom of the channel.

It was estimated that the channels would have to be lengthened by about 90 feet, to a total length of 150 feet, in order to remove grit sizes between 100 and 65 mesh (0.15 and 0.21 mm). Although these dimensions are theoretically satisfactory, they are not very practical from the grit removal equipment standpoint. Extremely heavy equipment would be needed to collect and remove grit from a channel this long.

Item 1 in Table 29 shows the estimated cost for lengthening the existing grit chambers. This cost includes replacing all of the chain and bucket collection equipment in the existing channels with heavy-duty equipment. The estimate does not include a revision of the head works of the grit chambers where the grit is removed from the channel. Therefore, even if the channels are lengthened in accordance with this alternative, there will still be unsatisfactory operating characteristics. Modifications necessary to change this configuration of the chambers would greatly increase the cost estimate, thereby further reducing the desirability of this alternative.

Although the additional length to the grit removal channels would relieve the sedimentation basins of large volumes of sand and silt, no particular improvement to floatable materials would be accomplished. Therefore, two additional bays would be necessary in the primary sedimentation basins to reduce the overflow rate and to improve floatable grease removal. Each bay is equivalent to two channels, 17.5 feet wide by 180 feet long. These additional bays would reduce the peak flow overflow rate from 4,600 gpsf per day to 3,800 gpsf per day, theoretically improving removals by as much as 20 percent.

Although the capital cost is attractive, this alternative for improving the grit handling capability of the plant is not recommended. The inherent problems of grit removal from the lengthened channels, the uncertain results and complexity of such a design, and interference with normal plant operation during construction were the major arguments against its selection.

TABLE 59

ESTIMATED CONSTRUCTION COST OF ADDITIONS AND REVISIONS TO THE WATER POLLUTION CONTROL PLANT*

1. Enlarge existing grit chambers**		
a. Grit chambers		
(1) Structural concrete and earthwork		$ 440,000
(2) Mechanical equipment		350,000
(3) Electrical		50,000
(4) Relocate bypass pipes		160,000
	Subtotal	$1,000,000
b. Additional sedimentation basins		
(1) Structural concrete and earthwork		$ 630,000
(2) Mechanical equipment		190,000
(3) Electrical		115,000
(4) Piping		125,000
(5) Miscellaneous		80,000
	Subtotal	$1,140,000
	Total	$2,140,000
	Contingencies (25%) +	540,000
	TOTAL	$2,680,000
2. Add new grit removal facilities		
a. Aerated grit chambers		
(1) Structural concrete and aeration and grit removal equipment		$1,270,000
(2) Blowers, air headers and blower building		190,000
(3) Electrical		50,000
(4) Yard piping		50,000
(5) Miscellaneous		40,000
	Subtotal	$1,600,000
b. Additional sedimentation basins		1,140,000
	Total	$2,740,000
	Contingencies (25%) +	690,000
	TOTAL	$3,430,000
3. Modify sedimentation basins and add sludge handling equipment		
a. Modifications		
(1) Structural concrete and earthwork		$ 710,000
(2) Mechanical equipment		630,000
(3) Electrical		130,000
(4) Piping and valves		160,000
	Subtotal	$1,630,000
b. Additional sedimentation basins		1,140,000
	Total	$2,770,000
	Contingencies (25%) +	690,000
	TOTAL	$3,460,000

*Estimating prices based on 1972 costs (ENR Index E.F. = 1900).

**Estimate does not include revisions to grit handling equipment at the head-end of the chambers.

Add New Grit Removal Facilities

This alternative plant improvement would involve constructing additional or substitute grit removal facilities just north of the present grit chambers. Two arrangements were considered:

1. Heavy-duty detritus tanks
2. Aerated grit chambers.

Heavy-Duty Detritus Tanks. As in the case of extending the existing grit chambers, the detritus tank installation was sized for removing grit sizes ranging between 100 and 65 mesh, based upon the manufacturer's recommendations. This removal was based on an average storm flow condition of 256 mgd, which was twice the design peak dry weather flow rate of 128 mgd. The selected configuration consisted of two 65-foot square units constructed in parallel.

The installation would include two 65-foot square, heavy-duty detritus tanks with integral grit classifiers. It would require the relocation of the existing storm water bypass which could be reconstructed as a part of the new grit facilities. As in the case of lengthening the existing grit chambers, two bays would be added to the sedimentation basins. The cost estimate for this alternative was similar to that for aerated grit chambers and was not included in Table 29.

The advantages of this alternative are that the construction would not disrupt normal operations of the plant and the equipment would not have to be used continuously but could be used as a supplemental grit removal facility during storm conditions. The disadvantages are that the bypass must be relocated and experience with the standard designs of such tanks has indicated some deficiencies in hydraulic distribution and resulting grit removal characteristics. Although the estimated cost is attractive, the uncertainties introduced from experience override the advantages. Therefore, this alternative is not recommended.

Aerated Grit Chambers. To be applicable as an alternative improvement to the plant for improving grit removal and handling capability, aerated grit chambers must be designed for the maximum wet weather flow rate. If this flow rate is not used in the design, the grit chambers will remove too little grit during the peak flow conditions. If it is used during dry weather flow conditions, the system would ensure particularly good removals.

The preliminary design consists of four tanks 60 feet long, 18 feet wide, and 15 feet deep. Because large volumes of grit will accumulate in relatively short periods during storms, a clam-shell type of grit removal mechanism is recommended.

Item 2 in Table 29 shows the estimated cost of the proposed aerated grit chambers. The estimate includes necessary air diffusers, air

headers and blowers together with clam-shell grit removal equipment, all necessary yard piping, structures, and excavation. It does not include a superstructure over the grit chambers but does include a separate structure for the blower equipment. Although the cost of the blower building is included in this estimate, it could possibly be included in the blower building for future secondary facilities.

As in the preceding alternatives, aerated grit chambers will not effectively remove floatable materials; therefore, the cost of additional sedimentation basins was included. The estimated cost is identical to that used previously.

This plan would be applicable to future conversion of the plant to an activated sludge system because sufficient grit would be removed to permit continuation of the present method of handling the primary sludge, i.e., digestion without sludge degritting. Waste-activated sludge could be thickened separately by flotation and combined with the primary sludge either before or after digestion. If sludge incineration is considered, the primary sludge should be sufficiently free of grit to permit vacuum filtration without further degritting.

These aerated grit chambers would permit year-round operation since they have substantially more detention time during the normal dry weather flow. If it were considered desirable to have all removal and handling facilities in one location, the floors of the existing grit chambers could be raised sufficiently to increase the sewage velocity and prevent settling of grit. This modification may actually be necessary to preserve the hydraulic capacity through the system without further structural and mechanical revision. Because the future plans for expansion at the plant have not been completed, further detailed conclusions could not be drawn.

This alternative has merit and should be considered equally with the following alternative which involves modifications to the sedimentation basins. The final selection between these two alternatives for recommentation should be based both on capital and operating costs and overall compatibility with the plans for future plant expansion.

Modify Sedimentation Basins and Add Sludge Handling Equipment

The merits of this plan are based on the probability that the water pollution control plant will be expanded to include secondary treatment and that future sludge handling will include dewatering and incineration for disposal in lieu of digestion. It was further assumed that waste-activated sludge would be thickened separately and then blended with primary sludge for disposal. In preparation for dewatering, particularly by centrifuging, it would probably be desirable to degrit the primary sludge. This alternative improvement therefore is presented to take advantage of that probable need for primary sludge degritting. Figure 48 is a schematic flow diagram for this alternative improvement.

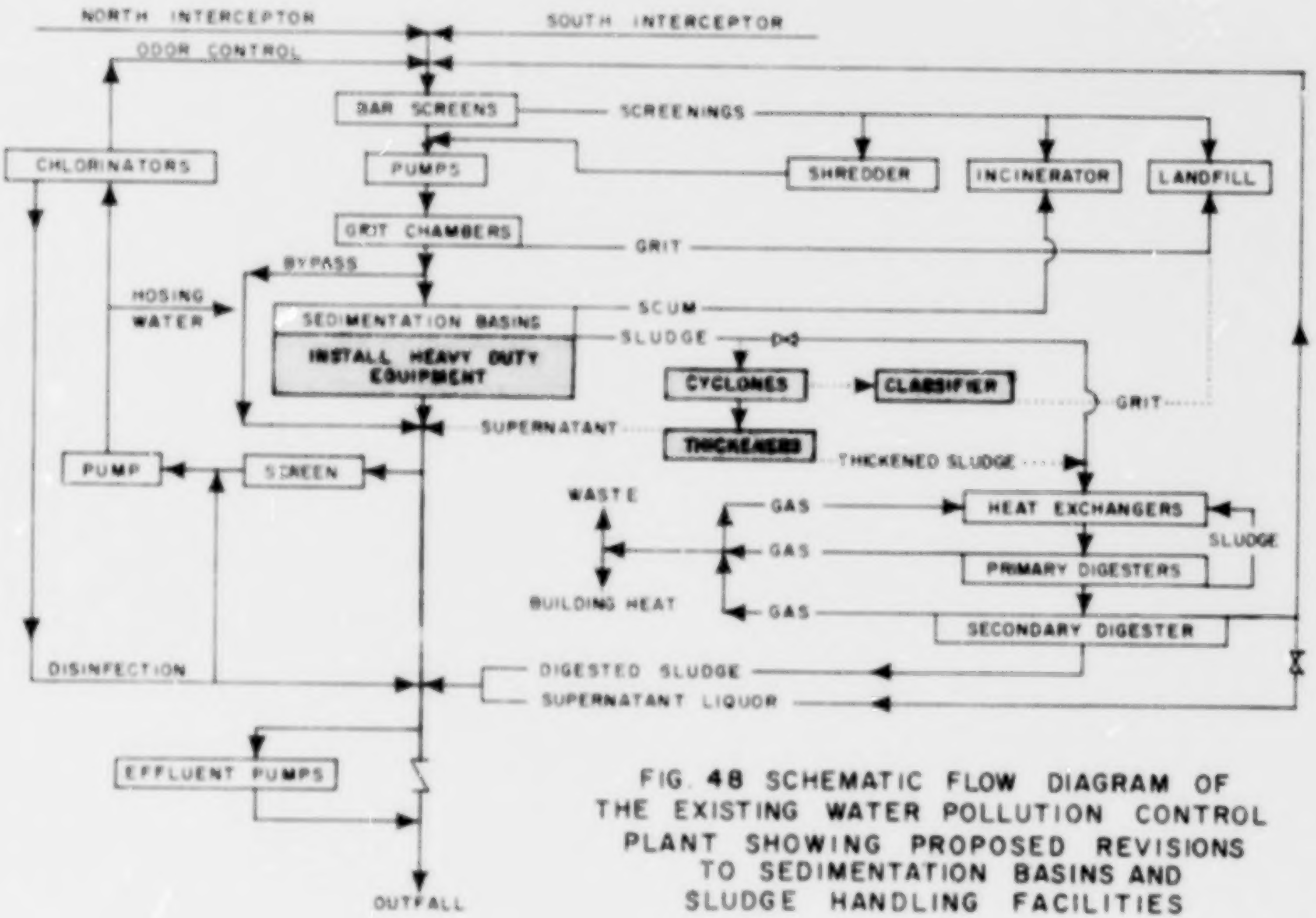

FIG. 48 SCHEMATIC FLOW DIAGRAM OF THE EXISTING WATER POLLUTION CONTROL PLANT SHOWING PROPOSED REVISIONS TO SEDIMENTATION BASINS AND SLUDGE HANDLING FACILITIES

The chain and flight collectors located in the sedimentation basins would be fitted with two-speed motor drives, and the cross-collectors would be replaced by heavy-duty, cross-collector equipment. These revisions would permit the operator to increase the speed of the chain and flight collectors, hence preventing excessive accumulation of sand and silt during periods of infiltration. In order to provide adequate capacity during storm flows, another complete bay would be added to the sedimentation basin. The sludge from each sedimentation channel would be pumped by individual pumps to cyclone-type degritters. Each cyclone would be sized to receive the discharge from two sludge pumps; therefore, six cyclones would be necessary. Based upon a solids concentration of less than 0.5 percent during dry weather conditions and approximately 1.0 percent during wet weather conditions, the cyclones would receive 600 gpm of flow. Each cyclone would have its own grit classifier. Grit from the classifiers would be discharged to a common conveyor and then to a truck or other container for ultimate disposal.

Because the sludge would be pumped from the sedimentation basins at low solids concentrations, sludge thickening would be necessary before pumping to the digesters, probably by the gravity thickening method. For this reason, gravity thickeners were sized and included in the equipment estimate. The thickened sludge would be pumped either to dewatering facilities for disposal by incineration or to the existing digesters for biological degradation.

Item 3 in Table 29 indicates the estimated cost for mechanical and electrical equipment, structures, and piping necessary for the sedimentation basin modifications and sludge handling equipment.

The advantages of this alternative are that 1) the scheme can be incorporated in the planning for future improvements to the plant, 2) the existing bypass channel would not be relocated, and 3) the equipment would be usable and applicable throughout the year. The disadvantage is the cost for operation and maintenance of the mechanical equipment involved. However, most of this equipment may become necessary in the near future. It was therefore concluded that this plan for improving the grit handling capability of the plant should be considered as an equal alternative to aerated grit chambers, hence should be selected on the basis of compatibility with plans for future plant expansion. Because the cost estimates are almost identical, either alternative could be chosen without affecting the combined cost estimates for selected combinations of component solutions.

SELECTION OF ALTERNATIVE COMBINATIONS OF COMPONENT SOLUTIONS

Drawing on the conclusions and cost estimates of the various component solutions in the preceding discussion, several alternative combinations were selected as being applicable to the problems of infiltration in the East Bay Area. It should be realized that each of these alternative solutions will allow some increase in flow during the storm or wet

weather conditions. The objectives in formulating the alternative solutions were:

1. To reduce infiltration or otherwise provide containment of the volume of water that enters the sewer during wet weather.

2. To provide sufficient treatment of any overflows to prevent pollution of the receiving waters.

3. To reduce greatly or eliminate bypassing at the water pollution control plant.

Using the objectives outlined above, the following combinations of component solutions were selected for consideration and comparison:

Alternative No. 1

1. Complete the sewer separation program.

2. Provide a series of holding tanks at strategic locations.

3. Provide plant improvements to prevent bypassing.

4. Locate and disconnect catchbasins presently connected to the sanitary sewers.

Alternative No. 2

1. Complete the sewer separation program.

2. Provide plant improvements to prevent bypassing.

3. Locate and disconnect catchbasins presently connected to the sanitary sewers.

4. Conduct sewer repairing and sealing program for all the remaining sanitary sewers.

Alternative No. 3

1. Complete about 50 percent of the remaining sewer separation program.

2. Provide plant improvements to prevent bypassing.

3. Locate and disconnect catchbasins presently connected to the sanitary sewers.

4. Provide treatment of overflows.

5. Eliminate sewer system bottlenecks to prevent intersystem overflows.

Alternative No. 4

1. Complete sewer separation program.

2. Provide plant improvements to prevent bypassing.

3. Locate and disconnect catchbasins presently connected to the sanitary sewers.

4. Provide treatment of overflows.

5. Eliminate sewer system bottlenecks to prevent intersystem overflows.

Table 30 summarizes the combined construction cost estimates for each of the alternative solutions, including an addition to cover administrative and engineering costs. Alternative No. 3 had the lowest capital cost. This alternative was based upon completing only about one-half of the remaining sewer separation program and installing overflow treatment facilities. The bulk of the combined sewers to be separated were assumed to be located in the commercial-industrial areas of the city of Oakland.

Alternative No. 1 had the second lowest capital cost. As discussed earlier in this section, the components of Alternative No. 1 (holding tanks) would be applicable to all storms that produce a typical rise and fall in flow rate, allowing the peak rate to be dampened by storage. However, under conditions of extended rainfall, storage would not adequately reduce the peak flow rate. In fact, the holding basins might become somewhat of a bottleneck in the system under those conditions. Because of this inherent weakness, it was concluded that Alternative No. 1 would not satisfy the needs of the East Bay Area.

Alternative No. 4 had the third lowest estimated cost. This alternative was similar to Alternative No. 3, except that all the remaining combined sewers were to be separated. This would permit the elimination of two overflow treatment facilities. The reduced treatment cost would not offset the increase in construction cost for the new sewers; hence only partial separation would be more economical as indicated in Alternative No. 3.

Alternative No. 2 was found to have the highest estimated capital cost. It should be noted, however, that this alternative would not add to the overall operation and maintenance costs for sewage collection in the East Bay Area. On the contrary, it would tend to reduce the annual maintenance cost because:

1. Less maintenance effort would be required during wet weather.

2. The frequency of sewer cleaning would be reduced in some areas.

TABLE 30

ESTIMATED CONSTRUCTION COST OF ALTERNATIVE COMBINATIONS OF COMPONENT SOLUTIONS*

Item description	Alternative No. 1	Alternative No. 2	Alternative No. 3	Alternative No. 4
1. Sewer separation	$19,250,000	$19,250,000	$ 9,600,000	$19,250,000
2. Provide a series of holding tanks	19,700,000	--	--	--
3. Provide plant improvements	3,430,000	3,430,000	3,430,000	3,430,000
4. Locate and disconnect catch-basins on sanitary sewers	400,000	400,000	400,000	400,000
5. Conduct sewer repairing and sealing program	--	58,800,000	--	--
6. Provide treatment of overflows	--	--	11,900,000	9,200,000
7. Eliminate sewer system bottlenecks	--	--	11,270,000	11,270,000
Subtotal	$42,780,000	$81,880,000	$36,600,000	$43,550,000
8. Administration and engineering (15%)	6,420,000	12,280,000	5,490,000	6,530,000
TOTAL CAPITAL COST	$49,200,000	$94,160,000	$42,090,000	$50,080,000

*Estimating prices based on 1972 values (ENR Index S.F. = 1900), and include an allowance for contingencies.

3. No new treatment equipment would be added throughout the area; hence there would be no additional maintenance personnel and power costs (both for power consumption and minimum demand charges).

4. The inevitable sewer replacement program, such as that presently adopted in Oakland, could be postponed.

A brief review of the relative operation and maintenance costs for these alternatives was made, and it was found that they were not great enough to change the relationship of the alternatives.

A definite factor in the selection of any one of the preceding combinations of component solutions involves the question of equality of results. Probably Alternative No. 2 would produce the best overall results because all flows could be directed through the treatment facilities at the plant. This assumes that a sewer repairing and sealing program, together with sewer separation and additional grit removal facilities at the plant, would sufficiently reduce the peak flow rates and the grit problems so that the sewage would receive at least primary treatment and possibly some could receive secondary treatment (split flow basis). The complete accomplishment of a sewer sealing program could require from 10 to 15 years. Without this component, the remainder of the alternative cannot produce the desired results. Therefore, the disadvantages of this alternative, timing and excessive cost, prohibit recommendation as a feasible alternative.

Alternatives No. 3 and No. 4 would be about equal in their results. The determining factor is the ability of the treating device to accomplish the necessary removals of grease and similar floatable materials. In those areas south of the water pollution control plant, the measured grease concentration was 105 mg/L during wet weather flows. A tentative resolution was recently submitted to the City of San Francisco by the Regional Water Quality Control Board, setting forth requirements for discharges from several wet weather diversion structures in that city. The requirements stipulate that grease concentration not exceed 25 mg/L and settleable matter not exceed 1.0 ml/L/hr at any time, as measured in the waste stream. If this limitation would be placed on discharges in the East Bay Area, those facilities south of the plant would require 80 percent grease and settleable matter removals to be acceptable. However, facilities located north of the plant would require only sufficient treatment to maintain the present average grease concentrations but enough to meet the settleable matter limitation. An additional comment was included in the resolution to the effect that this limit is to be considered as a goal until the costs for accomplishing these ends can be assessed. Based upon the belief that the necessary requirements can be met with proper equipment selection and appropriate design, either Alternative No. 3 or No. 4 would satisfy the requirements and would be a good solution to the storm water infiltration problems in the East Bay Area.

The ability of Alternative No. 1 to satisfy the requirements is the most uncertain of the several alternatives. The steps by which this alternative would produce results are similar to those described for Alternative No. 2, i.e., elimination of overflows. Because this investigation was very broad and general in its approach and conclusions, an absolutely positive statement cannot be made about the effectiveness of the proposed holding basins. The basin configuration and location studied during this investigation indicated good results, except during extended storms, but a gross extrapolation of this solution to the total study area may be optimistic. Also, the fact that extended storms cannot be effectively coped with by this design is an indication that positive results are not assured. Therefore, this alternative could not be recommended without additional extensive study.

On the basis of the foregoing discussion, Alternative No. 3 is recommended to the six East Bay cities and to EBMUD, Special District No. 1 for serious consideration and additional study as a solution to the problems of storm water infiltration.

It should be realized that the adoption and effectuation of Alternative No. 3 would not eliminate the need for a long-term sewer repair and replacement program. Unless such a program is initiated and followed, the infiltration quantities could increase until even the facilities described as part of Alternative No. 3 would be inadequate.

TIME SCHEDULE FOR RECOMMENDED ALTERNATIVE

In view of the impending action of regulatory agencies, the time schedule that would be necessary to put Alternative No. 3 into effect is important. The time schedule that appears to be most realistic in terms of present conditions is shown in Table 31. This schedule assumes that very little time will elapse before the East Bay cities and EBMUD approve the plan outlined in Alternative No. 3 and take the necessary action to initiate the work. It was assumed that the cities would undertake the necessary surveys needed to outline the design work for completing sewer separation, disconnecting catchbasins, and eliminating sewer bottlenecks as soon as funds could be made available to the departments involved. The design and construction of these phases of the program could be conducted simultaneously, producing several contracts rather than only one large one.

The most time-consuming phase of the program would be the overflow treatment facilities. The time schedule was based on the assumption that EBMUD would take almost immediate action toward the initiation of design studies on the type of facilities needed to actually meet the anticipated water quality requirements. Financing for such a study could possibly be developed through federal or state aid programs. It was taken for granted that EBMUD, Special District No. 1 would be the agency involved directly in the construction and operation of these facilities because this is presently its function in the East Bay sewage collection and treatment system.

TABLE 31

TIME SCHEDULE FOR EFFECTUATION OF RECOMMENDED ALTERNATIVE

Item description

Acceptance of plan by participants

Developing financing

Complete sewer separation
  a. Location survey
  b. Engineering design
  c. Construction

Provide plant improvements
  a. Pilot plant studies, if required
  b. Engineering design
  c. Construction

Locate and disconnect catchbasins on sanitary sewers
  a. Location survey
  b. Engineering design
  c. Construction

Provide treatment of overflows
  a. Preliminary pilot plant studies
  b. Engineering design of one unit
  c. Construction of one unit
  d. Evaluation of first unit
  e. Engineering design of remaining units
  f. Construction of remaining units

Eliminate sewer system bottlenecks
  a. Location survey
  b. Engineering design
  c. Construction

Although preliminary studies would be undertaken for final selection of equipment, it would not be wise to design and construct all of the proposed facilities until after one unit has been put in operation and fully evaluated. Unfortunately, this extends the time of completion substantially.

The improvements to the water pollution control plant would be most efficiently accomplished by being constructed as part of, or immediately after, any near-future revisions to the plant. For this reason, initiation of this phase of the program was shown as beginning about two years after submittal of this report. In general it appears that the entire program could be completed in about seven years.

MISCELLANEOUS RECOMMENDATIONS

During the conduct of this study two recent developments were considered to have promise for some of the problems in the East Bay Area: friction reducing chemicals and special pipe designed for relining existing pipes.

The addition of certain polymers to waste water in sewers has been shown to be effective in reducing friction and hence increasing the effective capacity of the sewer. Dosage rates in the range of 50 to 150 mg/L were found necessary to increase the apparent capacity by 100 percent. The techniques for feeding and handling the volumes of polymer necessary to be effective in a large installation have not been fully developed. Therefore, the use of this technique would actually be a test installation and as such might justify federal or state funds as a process demonstration.

Although other locations exist, one specific trunk sewer that would benefit from the application of polymers is the Fifth Street sewer, i.e., the one that runs along the south shore of Lake Merritt. At a point near the interceptor a definite constriction occurs. This bottleneck apparently causes surcharging in the sewer, the lifting of manhole covers, and overflowing. Therefore, further study of the interim use of polymers is recommended until a more permanent correction can be made.

A glass-fiber reinforced plastic pipe has been developed and used to reline older pipes, particularly for the gas and chemical industries. Although certain material and installation problems would have to be resolved, this liner pipe appears to be a good way to repair sewers in place. It would be applicable to both street sewers and house connections either separately or together. The liner pipe is normally placed in the old pipe in a continuous length and in a flattened condition. The length could be as much as several hundred feet. The pipe is then inflated and cured with steam under pressure. The repair could be considered equivalent to, or better than, replacing the old sewer with new pipes, both from the standpoint of usable life and leakage rates. The pipe walls are thin and would not materially reduce the hydraulic capacity; in fact, they might improve it because of the smooth interior.

It was concluded that this pipe could be used as a means to repair house laterals from a point near the building out to the sewer. This installation would entail digging one hole near the building and one at the connection to the street sewer; the remainder of the yard or paving would be left untouched. An appropriate cement would be used to seal each end of the liner to the existing and remaining sewer. The house lateral would then be a single continuous unit from the house out to the sewer, minimizing both infiltration and future maintenance problems.

An alternative to the above use would be to reline both the street sewer and the house laterals during one operation. This would result in an almost bottle-tight system. The sewer would be relined from manhole to manhole with a single length of pipe. Each house lateral connection would then be made by cementing a saddle type unit directly to the wall of the sewer liner. Because the liner pipe has structural integrity, both crushing and internal pressure, a short length of the new pipe could be exposed between the end of the old house lateral and the wall of the street sewer without concern.

It is therefore recommended that a study area be selected, perhaps one of those used in this investigation, and a controlled installation of this liner material be evaluated in conjunction with the manufacturer, American Cyanamid Co.[8] Two types of installations should be studied: one with only house laterals lined and another with both street sewer and house laterals lined. The objective of this test would be to determine the techniques of installation and the cost, as well as the possibility of relining in lieu of replacing sewers. Costs are suspected to be higher than for sewer sealing techniques but lower than for total replacement. Again, this further study would be in the category of research and development and might qualify for federal or state financial assistance.

Two other areas of interest were discussed in Section 10: improvements in sewer design and construction practices, and the application of the multipurpose concept. Both of these items merit the attention of the designers and should be used where applicable.

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[8]Mention of commercial products does not imply endorsement by Federal Water Quality Administration.

SECTION 12

ACKNOWLEDGMENTS

SECTION 12

ACKNOWLEDGMENTS

Execution of the project depended upon close cooperation among the City of Oakland; City of Berkeley; East Bay Municipal Utility District, Special District No. 1; and Metcalf & Eddy, Inc., contractors for the study. Metcalf & Eddy, Inc., gratefully acknowledges the contributions of these organizations and, in particular, the efforts of the following individuals:

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2. Harold M. Fones, Assistant Director of Public Works.
3. Henry McGraw, Public Works Maintenance Superintendent.
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1. Elmer E. Ross, Manager, Water Pollution Control Division.
2. John D. Foster, Senior Civil Engineer.
3. Edward Chow, Senior Sanitary Chemist.
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5. Robert Howard, Mechanic Foreman.

The following agencies were consulted while accumulating data concerning the locations and characteristics of current precipitation stations in the study area:

1. Alameda County Flood Control District, Hayward.
2. Alameda Naval Air Station, Alameda.
3. California Department of Water Resources, San Francisco Bay District, Vallejo.
4. East Bay Municipal Utility District, Oakland.
5. Oakland International Airport Meteorological Station, Oakland.
6. San Francisco Water Department, San Francisco.
7. U.S. Army Corps of Engineers, San Francisco District, San Francisco.

8. U.S. Weather Bureau Climatological Office, San Francisco.
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William G. Smith, Sanitary Engineer - Data Collection and Analysis Program
Charles A. Kohlhaas, Engineer - Economic Benefits Evaluation

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REFERENCES

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SECTION 14

GLOSSARY AND ABBREVIATIONS

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SECTION 14

GLOSSARY AND ABBREVIATIONS

GLOSSARY

AREAL RAINFALL DISTRIBUTION FACTOR - The ratio of the rainfall in a selected area to that measured at Oakland International Airport.

BUILDING SEWER - Same as house connection.

BYPASS - (noun) - An arrangement of pipe, conduit, gates, pumps and valves, whereby the flow may be passed around a hydraulic structure or treatment facility.
(verb) - The act of causing flow to pass around a hydraulic structure or treatment facility.

COMBINED SEWAGE - A sewage containing both sanitary or domestic sewage and surface water or storm water, with or without industrial wastes.

COMBINED SEWER - A sewer receiving both surface runoff and sewage.

DIRECT CONNECTION - Any opening, pipe, or other arrangement permitting storm water to directly enter the sanitary sewer.

DISPERSION - The mixing of polluted fluids with a large volume of water in a stream, estuary, or other body of water.

DOMESTIC SEWAGE - Sewage derived principally from dwellings, business buildings, institutions, and the like. (It may or may not contain groundwater, surface water, or storm water.)

DRY WEATHER FLOW - The combination of sanitary sewage, and industrial and commercial wastes normally found in the sanitary sewers during the dry weather season of the year, sometimes referred to as base flow.

EXTRANEOUS FLOW - That portion of the liquid carried in the sewer that is not normally classified as sanitary, commercial, or industrial waste or sewage. This investigation emphasizes the extraneous flows attributed to storm water infiltration.

GROSS INFILTRATION - The total infiltration entering a sanitary sewer by direct connections and by percolation through the soil.

GROUNDWATER (noun) - Subsurface water occupying the zone of saturation. In a strict sense the term applies only to water below the water table.

GROUNDWATER TABLE - The free surface of the groundwater, that surface subject to atmospheric pressure under the ground, generally rising and falling with the season, the rate of withdrawal, the rate of restoration, and other conditions. It is seldom static.

HOUSE CONNECTION - In plumbing, the extension from the building drain to the public sewer or other place of disposal.

INDUSTRIAL WASTE - The liquid wastes from industrial processes as distinct from domestic or sanitary sewage.

INFILTRATION - The process by which water, other than that classified as sanitary sewage or commercial and industrial wastes, enters the sewer system through any one of many different routes. In this investigation, infiltration includes all extraneous water during wet weather, i.e., groundwater and surface water from both direct and other connections.

INTERCEPTOR SEWER - A sewer which receives dry weather flow from a number of transverse sewers or outlets and conducts such waters to a point for treatment and disposal.

OVERFLOWS - The overflowing of trunk or interceptor sewers resulting from the combination of extraneous flows and normal flows that exceed their capacities. This investigation deals only with those overflows that occur during periods of wet weather and not those caused by power or mechanical failures.

OUTFALL SEWER - The outlet, structure, or sewer through which sewage is finally discharged.

PERCHED WATER - Groundwater that is separated from the main body of groundwater by unsaturated material.

PERCOLATING WATER - Subsurface water which passes through the soil or rock, along the line of least resistance and under the force of gravity, whose limits are not particularly defined by less permeable formations, and which does not of itself form a part of any definite body of subsurface water or flow in any subterranean channel.

PERCOLATION - The movement or flow of water through the interstices or the pores of a soil.

SANITARY SEWAGE - Same as domestic sewage.

SANITARY SEWER - A sewer which normally carries domestic sewage and into which storm water, surface water, and groundwater are precluded, so far as possible, unless intentionally admitted.

SEWER - A pipe or conduit generally closed, but normally not flowing full, for carrying sewage and other waste liquids.

SEWERAGE - A system of sewers and appurtenances for the collection, transportation, and pumping of sewage and industrial wastes.

STORM SEWER - A sewer which carries storm water and surface water, street wash and other wash water, or drainage, but excludes sewage and industrial wastes.

STORM WATER - Water resulting from precipitation which either percolates into the soil or runs off from the surface and is captured by storm drainage facilities.

SURFACE WATER - Rain water that falls onto the surfaces of roofs, streets, ground, etc. and not absorbed by that surface, thereby collecting and running off.

VADOSE WATER - Groundwater suspended or in circulation above the normal groundwater table.

ABBREVIATIONS

EBMUD - East Bay Municipal Utility District
ESSA - Environmental Science Services Administration
FWPCA - Federal Water Pollution Control Administration
FWQA - Federal Water Quality Administration, formerly FWPCA
USC&GS - United States Coast & Geodetic Survey

BOD - biochemical oxygen demand
cf - cubic foot
cfm - cubic feet per minute
cu m - cubic meter
COD - chemical oxygen demand
DO - dissolved oxygen
fpm - feet per minute
g - grams
gad - gallons per acre per day
gpd - gallons per day
gpm - gallons per minute
gpsf - gallons per square foot
lf - linear feet
mg - milligram
MG - million gallons
mgd - million gallons per day
mg/L - milligrams per liter
ml - milliliter
MPN - most probable number
ppb - parts per billion
psf - pounds per square foot
sq ft - square foot
sq in. - square inch
sq m - square meter
ss - suspended solids
tss - total suspended solids
tvs - total volatile solids
vss - volatile suspended solids

SECTION 15

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TABLE I-1

KEY TO GEOLOGICAL SYMBOLS ON FIGURE I-1

gb	Gabbro.
Jk	Knoxville Formation, shale, sandstone and minor conglomerate.
Kjfc	Chert and shale.
Kjfg	Greenstone.
Kjfm	Metamorphic rocks, undifferentiated.
Kjfs	Sandstone and shale, some chert, greenstone, serpentine, metamorphic rocks.
Kjm	Sandstone, shale, conglomerate.
Ko	Oakland Conglomerate.
Kr	Sandstone.
Ks	Shale.
Ku	Undifferentiated Upper Cretaceous rocks; may include unrecognized Eocene rocks.
Qac	Alluvium and colluvium.
Qaf	Artificial fill.
Qg	Unnamed gravel, sand and clay.
Qm	Merritt sand.
Qsl	San Antonio Formation lower member, gravel with silty clay matrix.
Qsu	San Antonio Formation upper member, clay, silt, sand and gravel.
Qtb	Material contemporaneous with Qtc and Qts deposited by streams flowing from Berkeley Hills.
Qtc	Temescal Formation, dark alluvium.
Qts	Alluvial material derived from the San Antonio Formation.
Qu	Undifferentiated Quaternary deposits.
Sp	Serpentine.
Tb	Briones Sandstone.
Tbp	Bald Peak Formation, predominantly basalt.
Tc	Claremont Shale.
Tcc	Contra Costa Group, undifferentiated. Includes unnamed sedimentary rocks.
Tem	Markley Sandstone, predominantly clay shale.
Tes	Unnamed Eocene sandstone and shale.
Th	Hambre Sandstone.
Thl	Hercules Shale member.
Tk	Kirker Tuff.
Tku	Paleocene siliceous shale and Upper Cretaceous sandstone units, undifferentiated. Predominantly sandstone unit, with minor amounts of siliceous shale too small to be shown on map.
Tl	Leona Rhyolite.
Tm	Moraga Formation, basalt, andesite and intercalated clastic rocks.
Tmz	Martinez Formation, sandstone and siltstone.
Tn	Northbrae Rhyolite.
To	Oursan Sandstone.
Tor	Orinda Formation, conglomerate, siltstone, sandstone, claystone.
Tp	Siliceous Shale.
Tr	Rodeo Shale.
Ts	Siesta Formation, silty claystone and sandstone.
Tso	Sobrante Formation, sandstone and silt stone.
Tsp	San Pablo Formation, predominantly sandstone.
Tsr	San Ramon Sandstone.
Tt	Tice Shale.
----	Contact.
——	Fault.
······	Concealed fault.
?	Doubtful.

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FIG. I-1 GEOLOGICAL FORMATIONS OF THE EAST BAY AREA

SOURCE: RADBRUCH, D. AND CASE, J.E., "GEOLOGIC MAP AND ENGINEERING GEOLOGIC INFORMATION, OAKLAND AND VICINITY, CALIF." U.S. GEOLOGICAL SURVEY, OPEN-FILE REPORT, 1967

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FIG. I-2 LAND USE IN THE EAST BAY AREA

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FIG 1-2
(CONTINUED)

24B

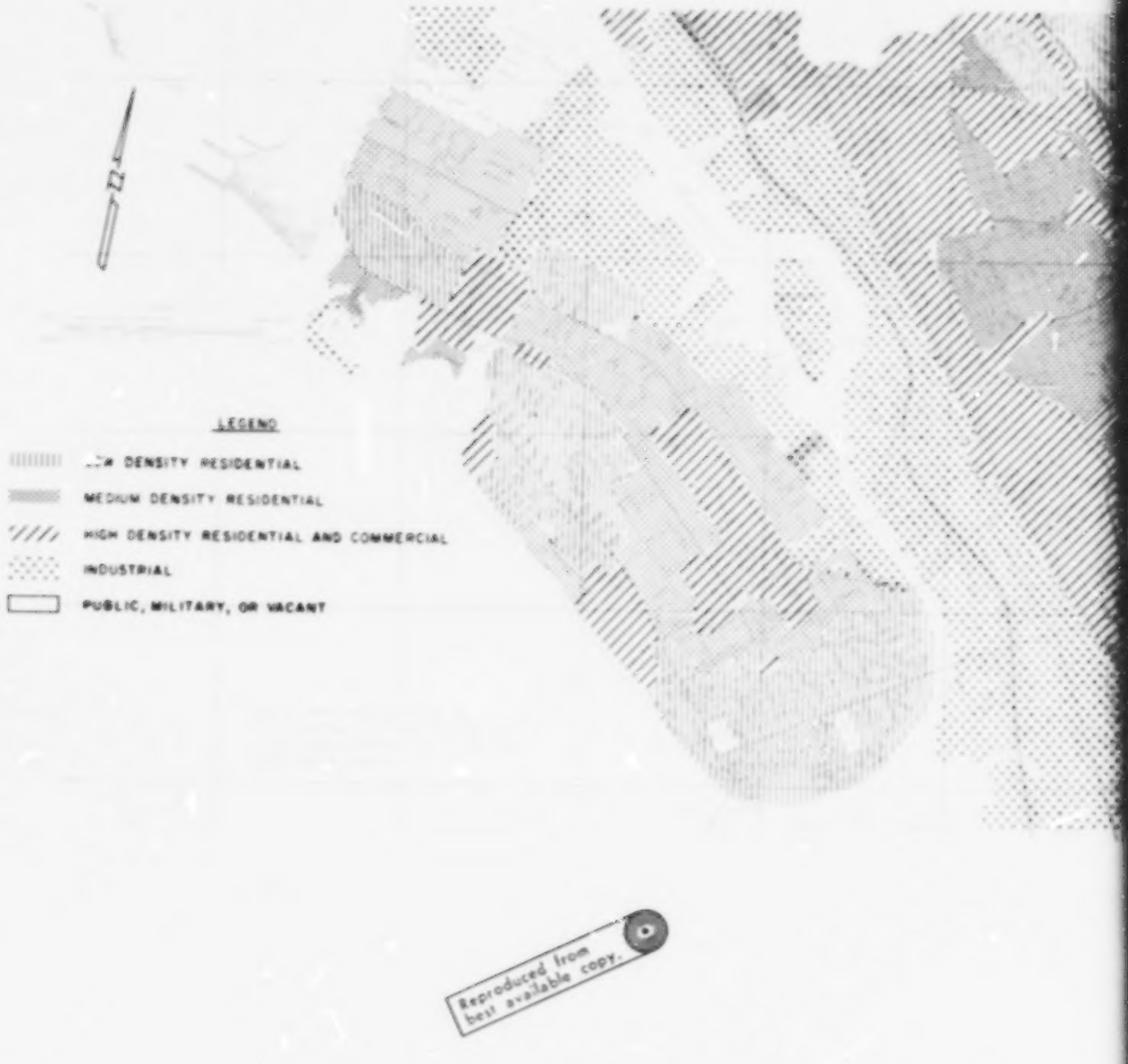

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FIG. I-2
(CONTINUED)

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FIG I-2
(CONCLUDED)

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FIG. I-3 SANITARY SEWER DRAINAGE DISTRICTS OF THE EAST BAY AREA

COPY OF SEWER SURVEY REPORT FOR KEITH STREET, BERKELEY

Submitted By Pacific N.W. Pipeline Survey Co., Inc., Vacaville, Calif.
July 22, 1968

REPORT　　　　Roll 21

This report covers the 10 inch sanitary sewer line on Keith between Euclid and Brett Harte Road in the City of Berkeley, as shown in the diagram on the cover page. The picture interval is 2 feet. The camera was pulled backward with the lens facing Manhole A.

All measurements should be made from 2 feet past the center of Manhole A.

The general condition of this line is poor. The pipe is relatively sound, but will take extensive help to bring up to standards, due to poor alignment and major offsets.

The grade is fair. A hillside installation is indicated by fast water, and is the only reason why the grade is acceptable. At 598 feet there is a pocket of water standing.

The alignment is very poor. While the line is normally quite curved in this installation, very abrupt changes are evident at numerous places, especially near the end of the run.

Side sewers are located at 301 RP, 339 RP and 372 1/2 R feet.

There are cracks at 3,49, 389, 417, 424, 457, 476 to 478, 493, 495, 497, and 554 feet.

Breaks are located at:
From 6 to 13 feet the pipe is broken, displaced, and crushed oval, from 128 1/2 to 130 1/2 the pipe is broken. Other breaks are at 161 1/2, 259, 312 1/2, 320, and 429 1/2 feet, where a chip is missing.

Joint separation were noted at:
15, 31, 75 1/2(major), 97, 105, 117 1/2(major), 141, 587, 589, 591, 596, and 609 feet.

Joint offsets were listed at:
4 1/2, 15, 31, 65, 70 1/2, 75 1/2(major), 81, 87, 99(major), 135, 113 (major,with void), 123(major), 133, 157, 295, 349, 415 1/2, 432 1/2, 596, and 609 feet.

There are intrusions at 75 1/2, 301(roots at side sewer, as well as at joint), 339(roots at side sewer installation), 429 1/2, 445, 457 1/2, 483, 485, 487, 565, 569 1/2, and 572 1/2 feet.

There is no infiltration apparent, but quite logical at bad joints, breaks, and intrusions.

Remarks: New pipe appears installed from 13 to 70 1/2 feet.

COPY OF SEWER SURVEY REPORT FOR KEELER STREET, BERKELEY

Submitted By Pacific N.W. Pipeline Survey Co., Inc., Vacaville, Calif.
April 19, 1968

REPORT

Roll 14

This report covers the 6 inch sanitary sewer line on Keeler between Marin and Forest Hill in the City of Berkeley, as is shown in the diagram on the cover page. The picture interval is 2 feet. The camera was pulled backward, with the lens facing manhole A.

All measurements should be made from 3 feet past the center of Manhole A.

The general condition of this line is fair.

The grade appears good, except for one dip between 308 and 316 feet.

Alignment: The pipe tends to wander, causing minor offsets.

Side sewers are located at 57 L, 104 L, 151 L, and 231 1/2 LP feet.

There are cracks visible at 87, 109, 111, 306 1/2, 315, and 331 feet.

Breaks: At 107 the spigot appears chipped. At 244 feet the pipe is crushed, displaced and oval. Also from 325 to 327 feet the pipe is crushed, displaced, and oval shaped.

Joint separations were noted at 242 1/2, 244 1/2, 306 1/2, and 329 feet.

There are joint offsets at 9, 11, 15, 46, 52, 60 1/2, 83, 91, 105, 113, 134, 153, 183, 199, 206, 300 1/2, 302 1/2, 303 1/2, 306 1/2, 318 1/2, 321, 323, 329, 331, and 344 feet.

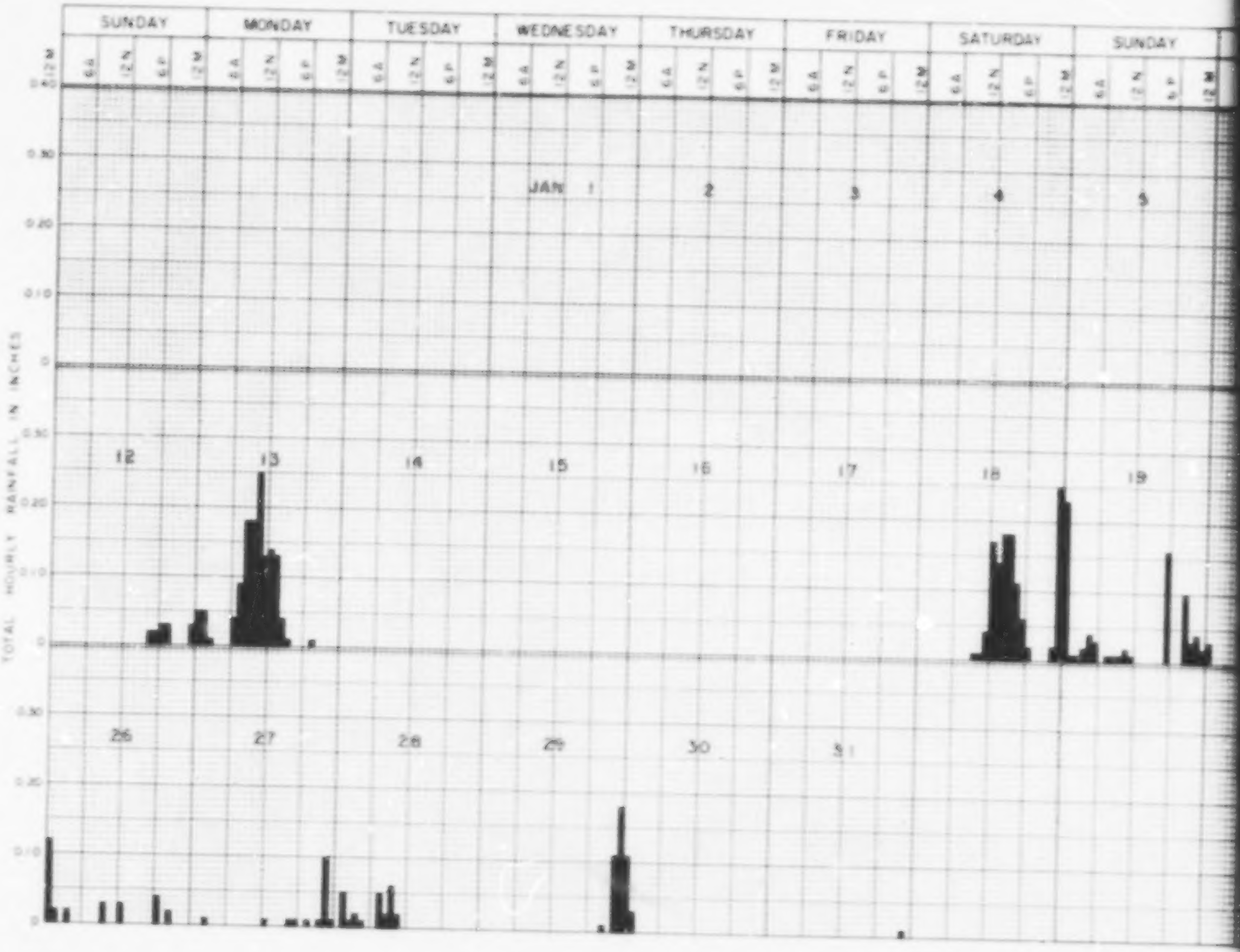
Root intrusions were listed at 11, 15, 29, 48, 57, <u>64 1/2</u>, 68, 111, 113, 115 1/2, 117 1/2, 119 1/2, 121 1/2, 123 1/2, 125 1/2, 127 1/2, 134, 189, 191, 193, 195, 197, 206, 238, <u>242 1/2</u>, <u>244 1/2</u>, 246 1/2, 250 1/2, 255, <u>257</u>, <u>259</u>, 265 1/2, <u>267 1/2</u>, 269 1/2, 271 1/2, 273 1/2, <u>276</u>, <u>278</u>, <u>280</u>, 282, 284, 286, 288, 290, 300 1/2, 325, and 329 feet. Massive ones are underlines.

Infiltration seepage is noticeable at most areas of intrusion.

The pipe appears out of round at 9, 11, 15, 17, 25, 46, 48, 59, 113, 115 1/2, 124, 148, 183, 185, 228, 242 1/2, 244 1/2, 246 1/2, 261, 278, 294 1/2, 331, and 344 feet.

Remarks: A root intrusion problem exists in this line. Broken pipe areas could completely fail at any time, causing stoppages. It is our opinion that the joints in this line are predominently faulty, due possibly to the type of sealant used.

251


SUNDAY
MONDAY
TUESDAY
WEDNESDAY
THURSDAY
FRIDAY
SATURDAY
SUNDAY
12M
6A
12N
6P
0.40
0.30
0.20
0.10
0
TOTAL HOURLY RAINFALL IN INCHES
JAN 1
2
3
4
5
12
13
14
15
16
17
18
19
26
27
28
29
30
31

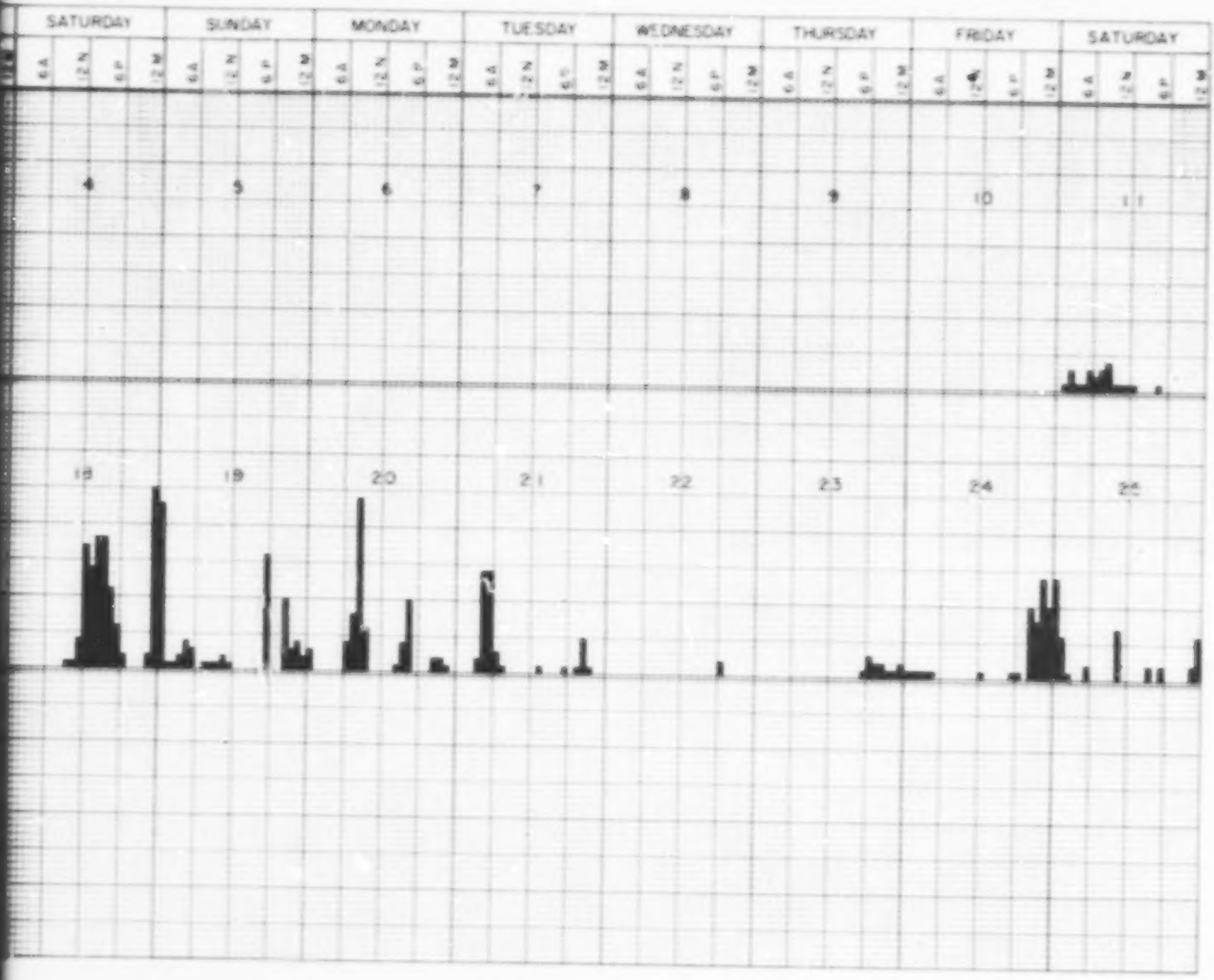
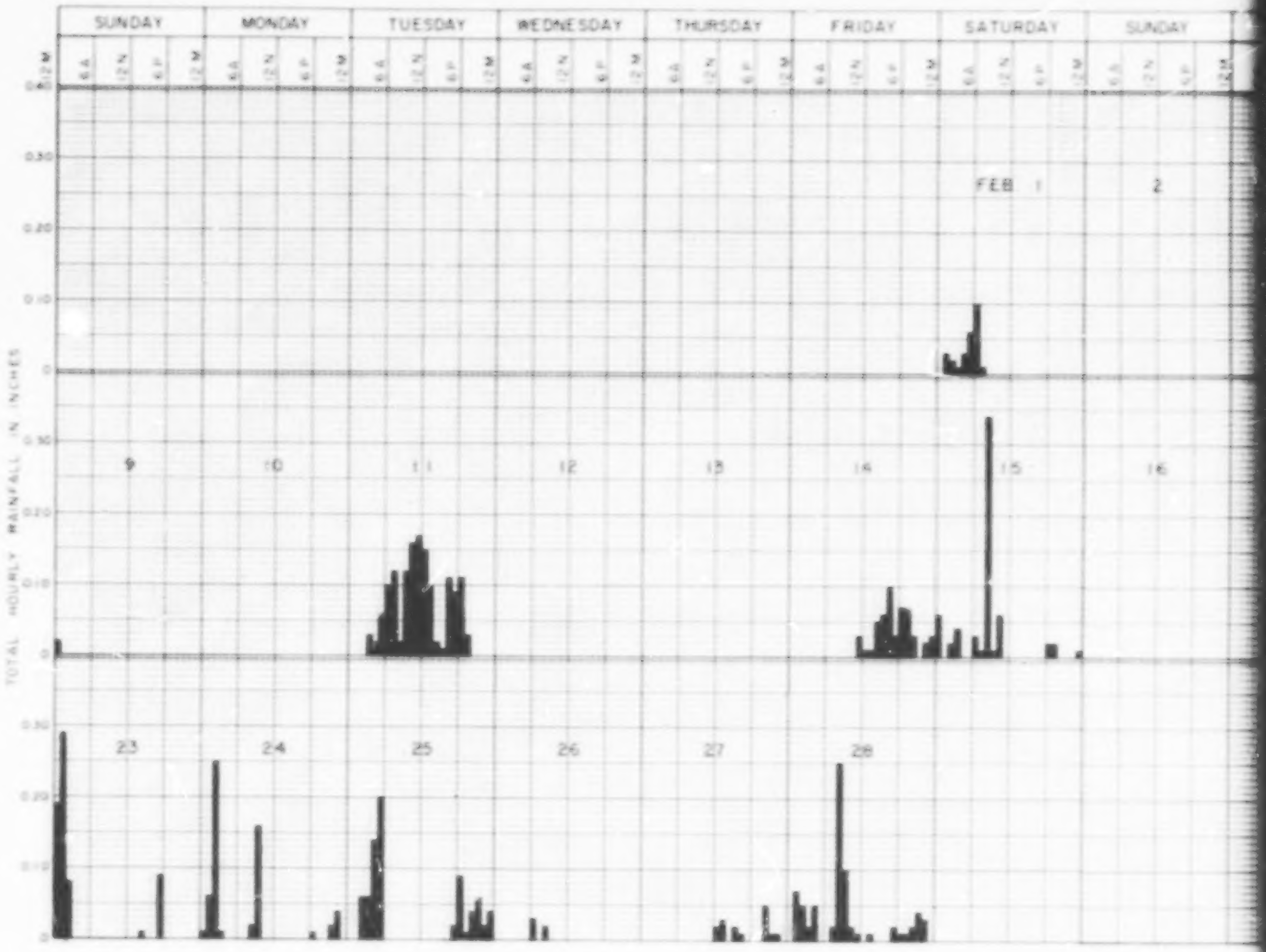


FIG. II - I HOURLY RAINFALL RECORD
JANUARY 1969
OAKLAND INTERNATIONAL AIRPORT


SUNDAY
MONDAY
TUESDAY
WEDNESDAY
THURSDAY
FRIDAY
SATURDAY
SUNDAY
FEB 1
2
9
10
11
12
13
14
15
16
23
24
25
26
27
28
TOTAL HOURLY RAINFALL IN INCHES
0.30
0.20
0.10
0

(254)

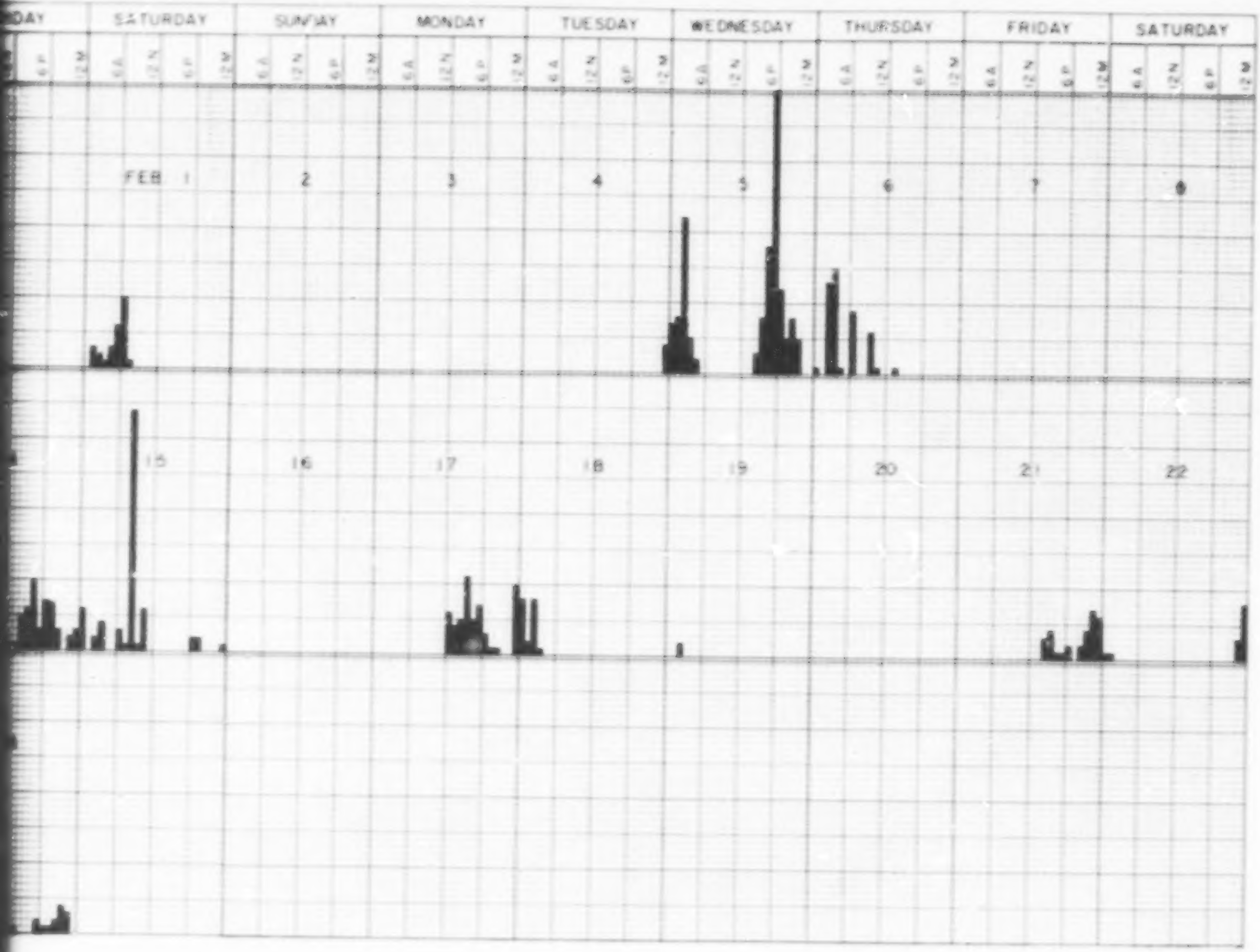


FIG II-2 HOURLY RAINFALL RECORD
FEBRUARY 1969
OAKLAND INTERNATIONAL AIRPORT

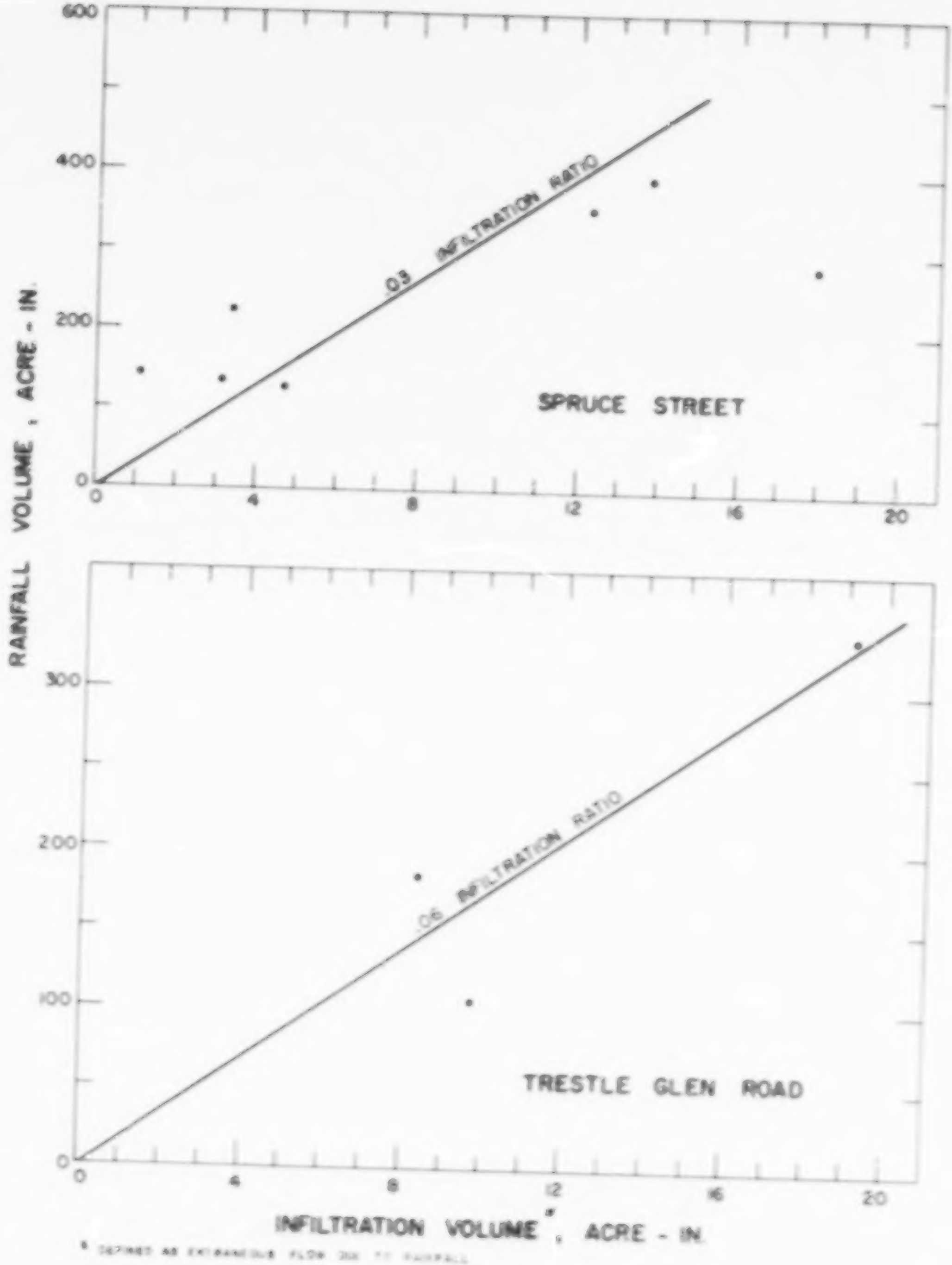


FIG. II-3 RAINFALL VOLUME VERSUS INFILTRATION VOLUME FOR STUDY SUBAREAS, 1968-69 STORMS

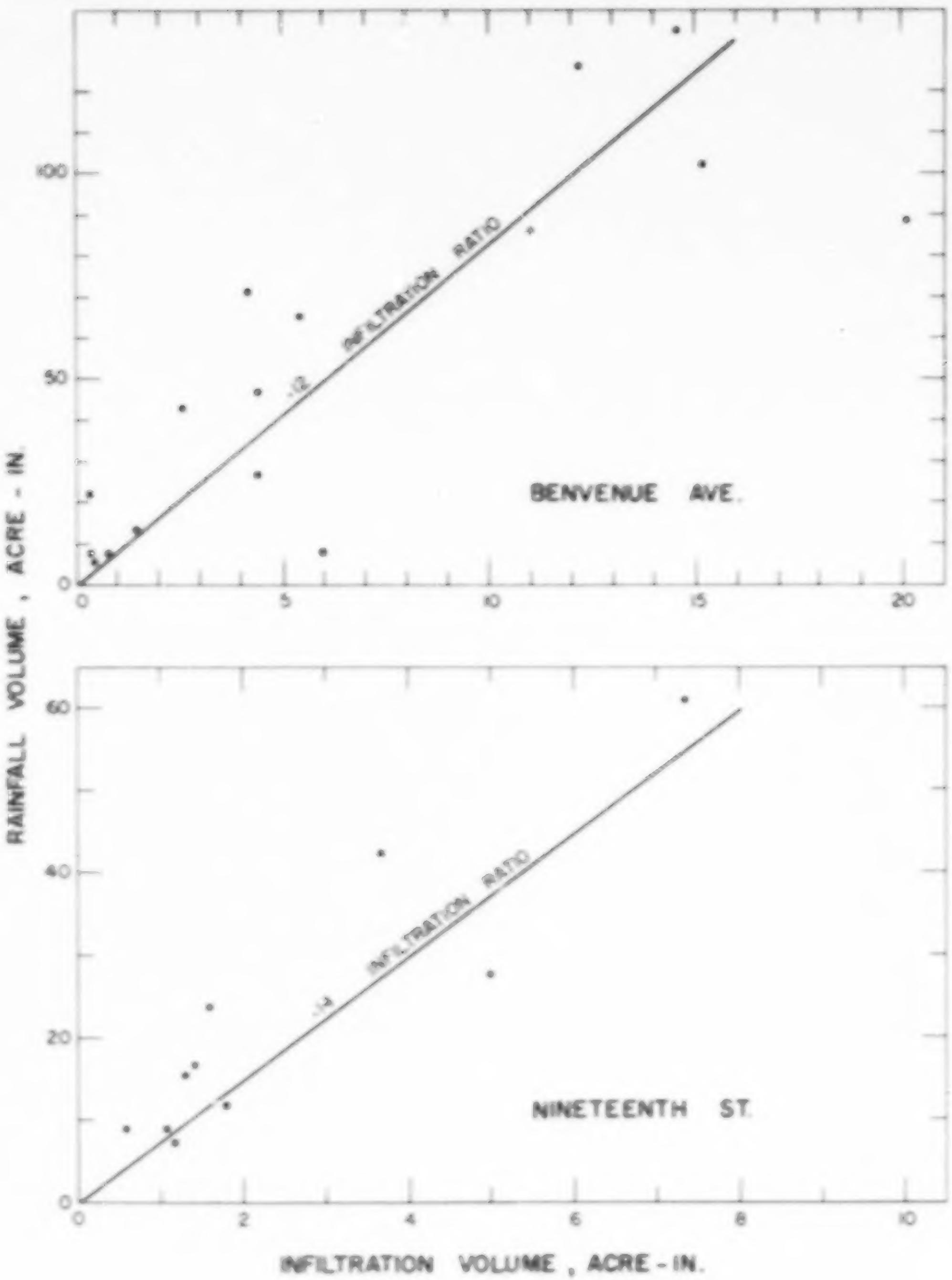


FIG. II-3 (CONTINUED)

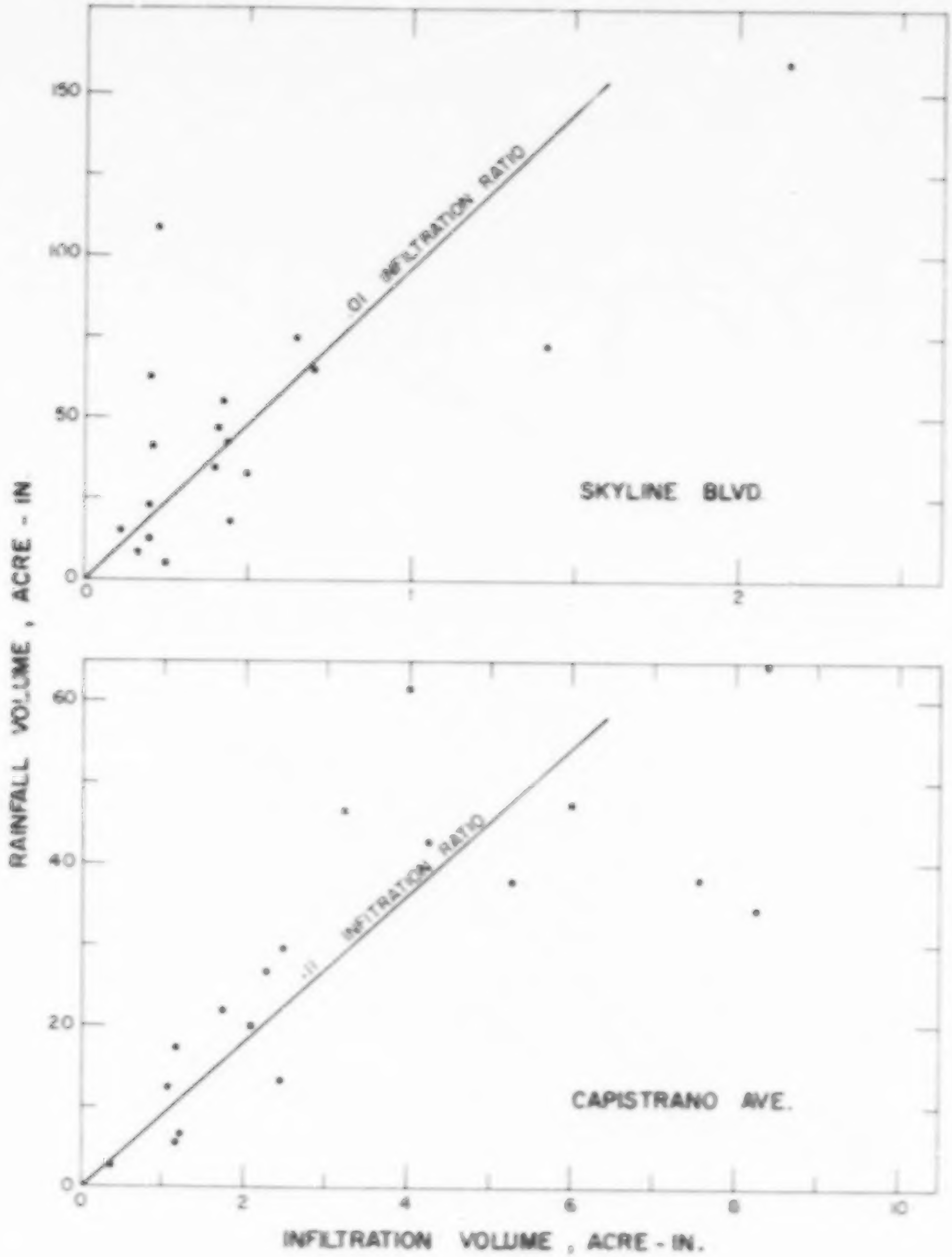


FIG. II-3 (CONTINUED)

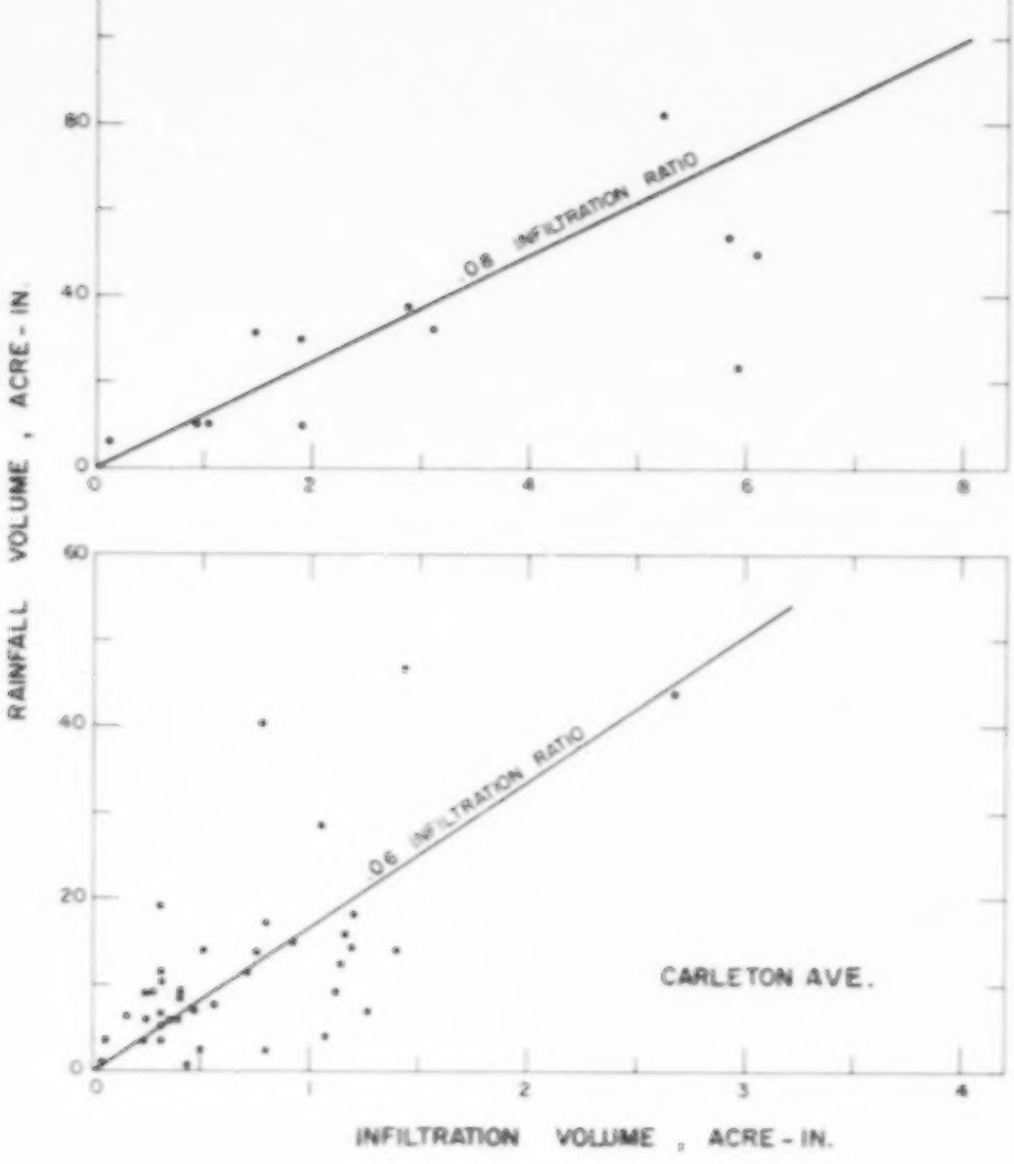


FIG. II-3 (CONTINUED)

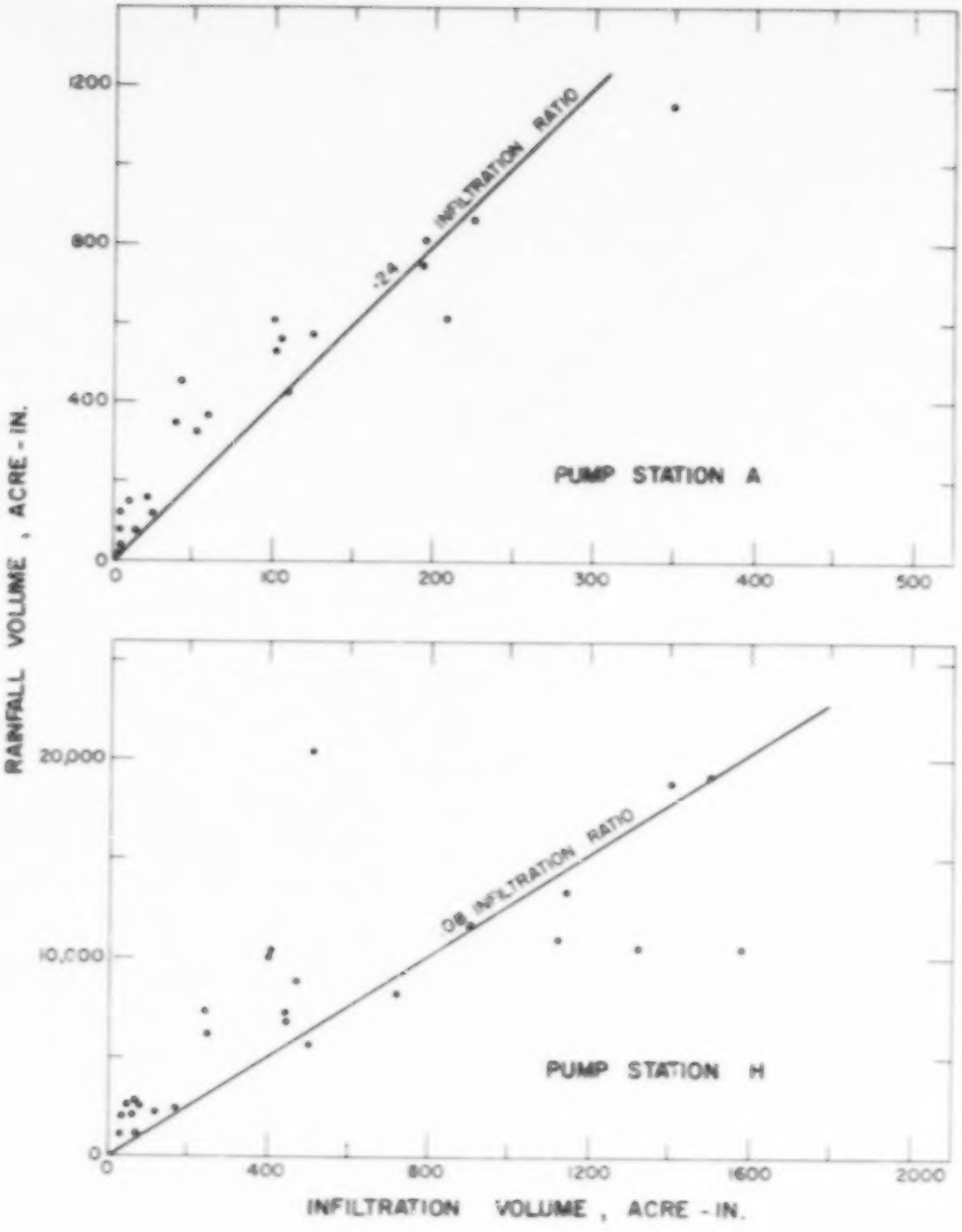


FIG. II-3 (CONCLUDED)

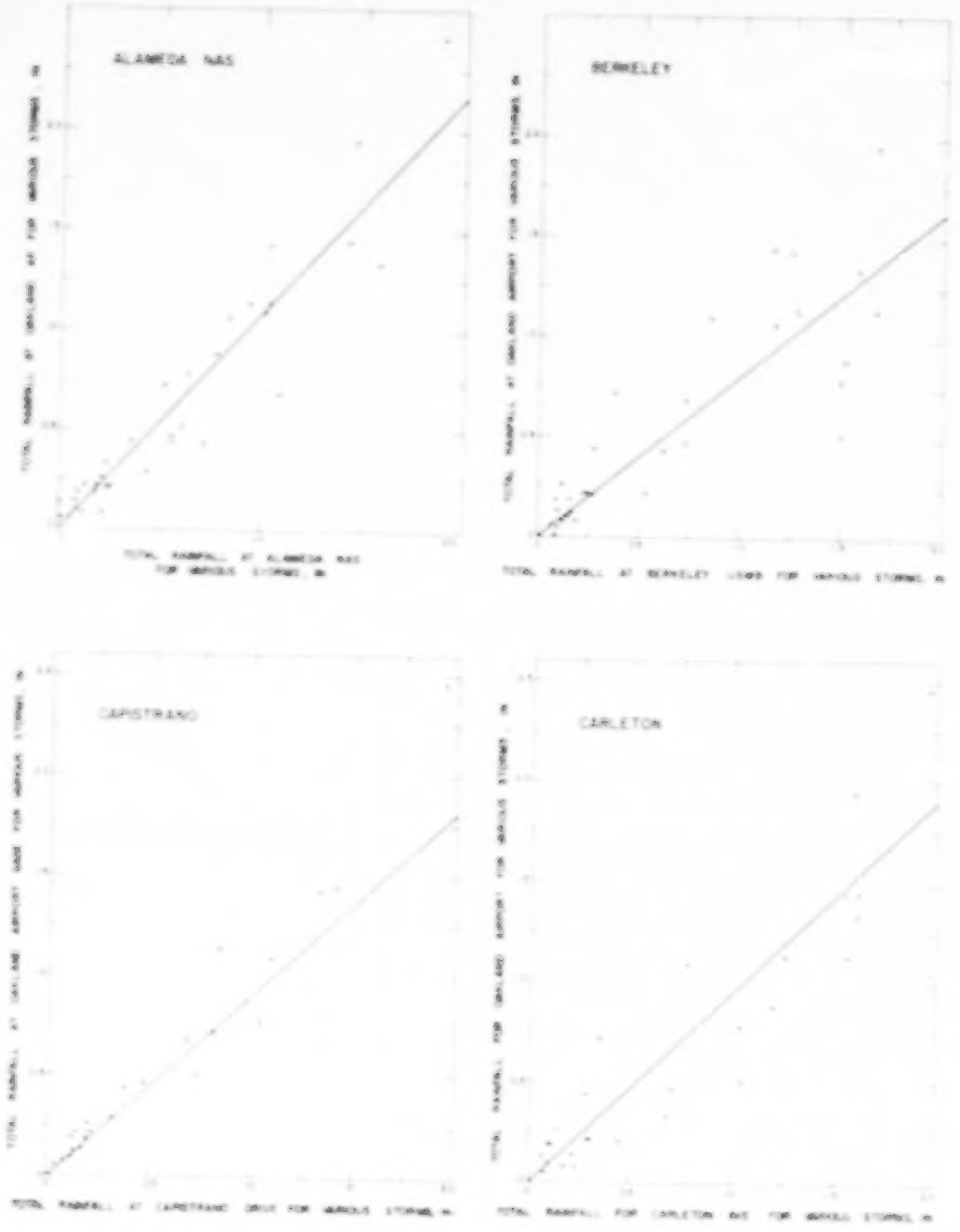


FIG. II-4 RAINFALL AT OAKLAND AIRPORT VERSUS RAINFALL AT OTHER LOCATIONS FOR 1968-69 STORMS

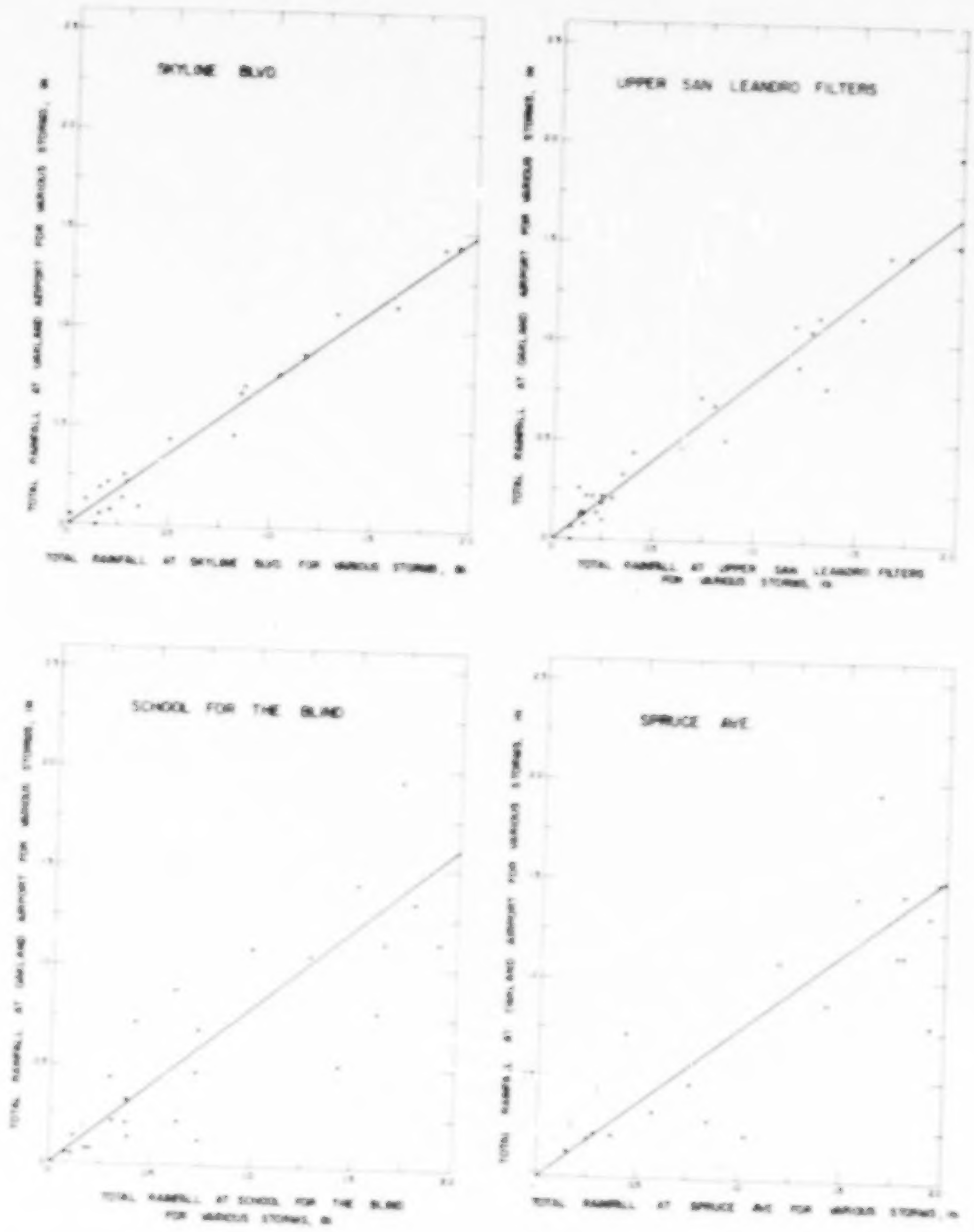


FIG. II-4 (CONTINUED)

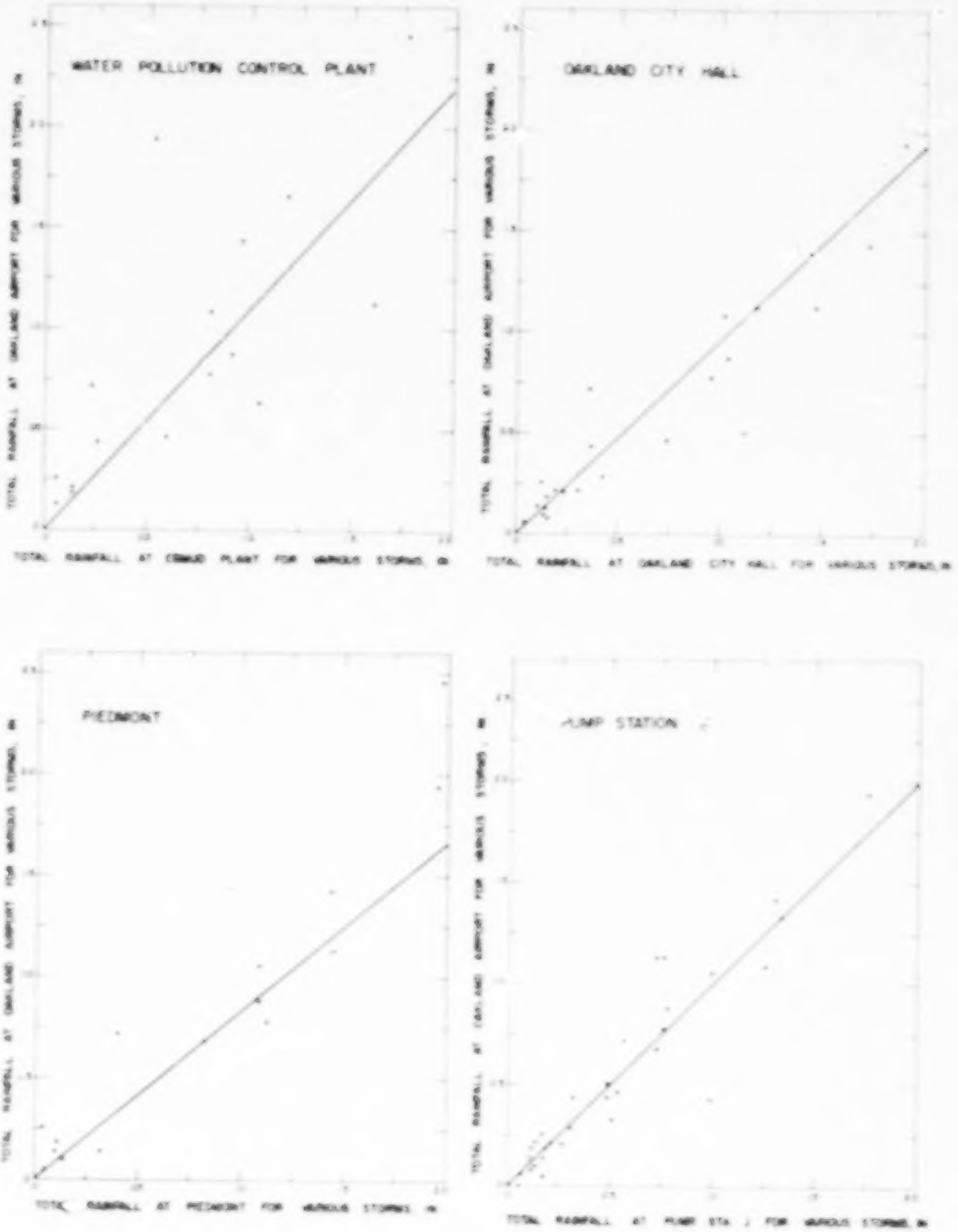


FIG. II - 4 (CONTINUED)

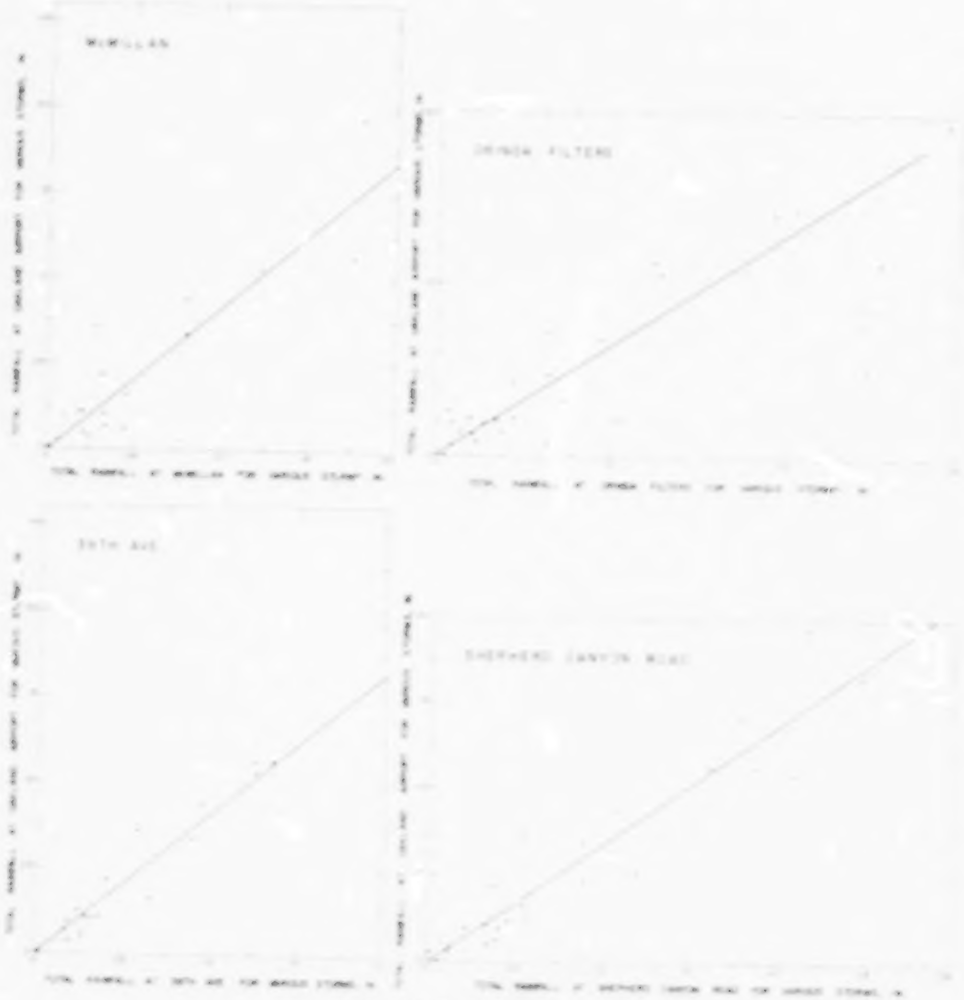


FIG II-4 (CONTINUED)

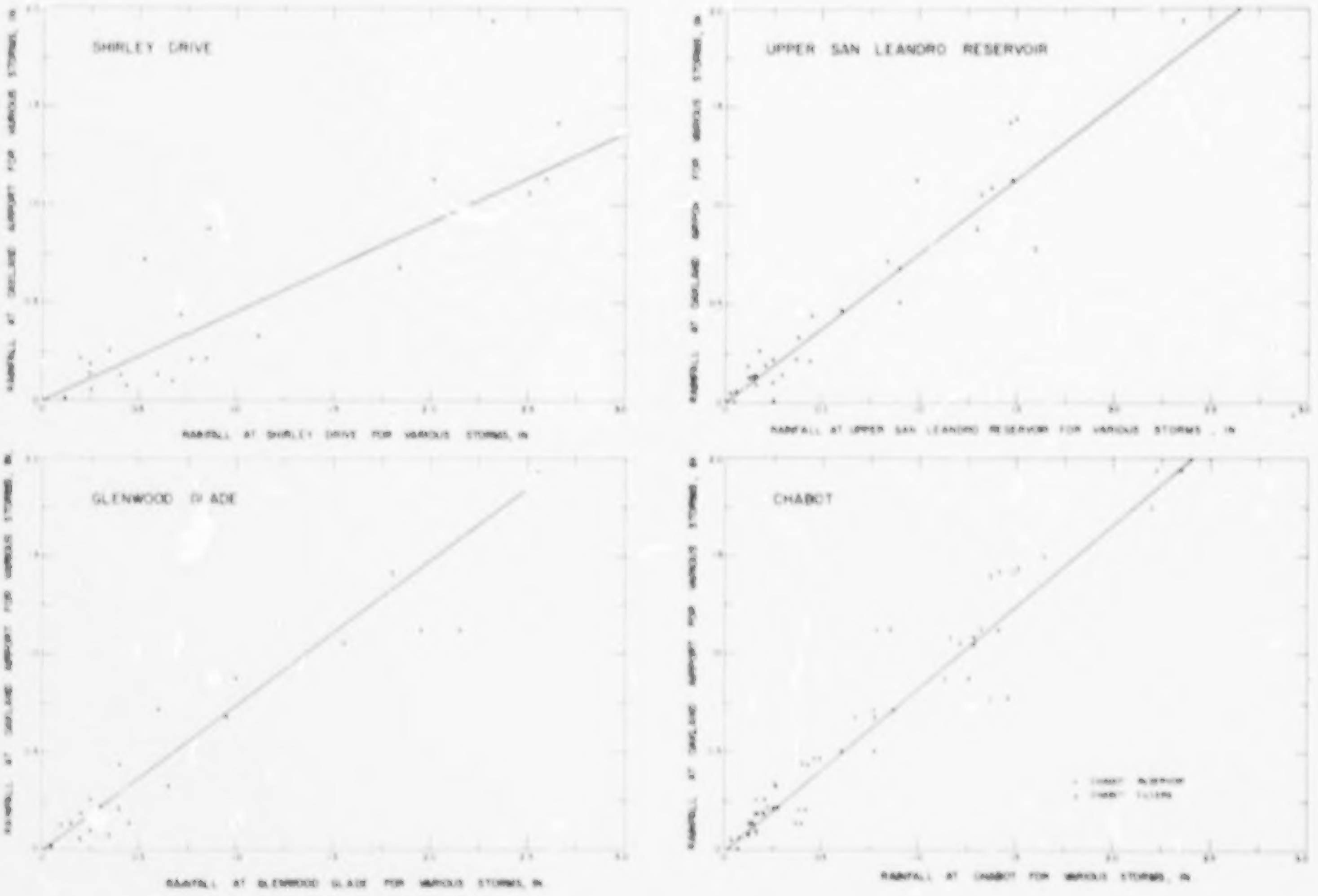


FIG. II-4 (CONCLUDED)

DESCRIPTION OF INTERSYSTEM OVERFLOWS AND CONNECTIONS

The following is a description of the six main types of intersystem overflows and connections found on the sanitary sewer systems of the various cities comprising EBMUD, Special District No. 1.

1. Improperly converted combined sewers. Intermittent reaches of old combined sewers are converted to separate storm and sanitary sewers, respectively. That portion of the sewer remaining was abandoned by plugging. Each end of the connecting reach of sewer was plugged at existing manholes. The pipe between the plugs was filled with sand. The sanitary portions of these sewers subsequently surcharged and, at times, overflowed onto the streets during periods of rainfall. Removal of the top few inches of both the plugs and the sand provided a relief overflow for the sanitary sewer to the storm sewers and stopped the overflowing onto the street.

2. Common manholes. Common manholes are used for both storm and sanitary sewers. At these locations, the manhole is on the storm line. The sanitary sewer is piped directly through the manhole with the invert of the sanitary line above the crown of the storm sewer. The upper one-fourth of the sanitary sewer pipe in the manhole is removed to provide access to the sanitary sewer and to act as a relief overflow during excessive sanitary flow.

3. Direct overflows to storm sewers. Direct overflows from sanitary sewers to storm sewers exist at manholes. These are generally located so that the sanitary sewer must surcharge to reach the invert of the overflow pipe. The invert of the overflow pipe is usually above the crown of the storm sewer. In cases where it is not, a flap valve is placed on the outlet of the overflow pipe to prevent storm flow from entering the sanitary sewer.

4. Direct connections of storm sewers to sanitary sewers. Direct connections from isolated storm sewers to sanitary sewers exist at several locations in the East Bay. In these cases, the storm sewers terminate at sanitary manholes. These provide for storm runoff from several catchbasins on each line to flow directly into the sanitary system.

5. Subsurface drains. Subsurface drains connected to sanitary sewers provide another method of entry for storm water. These drains are found primarily in the hillside areas of Oakland and Berkeley where excess groundwater seeps to the surface or causes slippage and movement of the soil mass. This land

movement causes other local problems, such as cracked, broken, or displaced sewers.

Subsurface drains are usually made of perforated steel pipe which collects groundwater and thereby relieves the soil pressure. These drains collect water continuously throughout the year, but the quantities increase during the rainy season.

6. Overflows to natural drainage channels. Overflows were found between sanitary sewers and natural drainage channels. Here, the sanitary sewer must surcharge before reaching the overflow connected directly to a nearby creek or lake. The surface runoff in the creek or lake dilutes and transports the overflowing sewage to the Bay.

TABLE III-1

REVISIONS TO WATER QUALITY OBJECTIVES FOR TIDAL WATERS INLAND FROM GOLDEN GATE WITHIN SAN FRANCISCO BAY REGION

Quality Parameter	Unit	[illegible]							
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]							
	January	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	February	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	March	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	April	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	May	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	June	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	July	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	August	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	September	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	October	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	November	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	December	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]		[illegible]							[illegible]

NOTES: a. [illegible]

[illegible]

b. [illegible]

c. [illegible]

d. [illegible]

e. [illegible]

SOURCE: State of California, Regional Water Quality Control Board, San Francisco Bay Region, [illegible] Revisions to Water Quality Objectives for Tidal Waters within the San Francisco Bay Region," [illegible]

TABLE IV-1

PRELIMINARY DESIGN DATA

WATER POLLUTION CONTROL PLANT REVISIONS

	Design flows Average	Peak
Sedimentation basin revisions and additional sludge handling equipment		
Plant flow	128	291
Sludge collector speed, fpm	3	12
Sludge pumps		
Number	10	10
Capacity, gpm	300	300
Solids concentration pumped, %	0.5-	1.0+
Cyclones		
Number	5	5
Capacity, gpm	600	600
Classifiers, number	5	5
Sludge thickeners		
Primary sludge to thickener, lb/day	176,000	376,000
Solids loading required, psf/day	20	40
Total area required, sq ft	8,800	9,400
Number of tanks	2	2
Tank dimensions		
Diameter, ft	80	80
Sidewater depth, ft	10	10
Area per tank, sq ft	5,000	5,000
Total tank area, sq ft	10,000	10,000
Solids loading, psf/day, average	17.6	37.6
Required total feed flow, mgd	4.32	4.32
Aerated grit chambers		
Flow, mgd	128	291
Number of tanks	4	4
Tank dimensions		
Length, ft	60	60
Width, ft	20	20
Average water depth, ft	14	14
Unit area, sq ft	1,200	1,200
Total area, sq ft	4,800	4,800

TABLE IV-1 (continued)

	Design flows	
	Average	Peak
Unit volume		
cf	20,400	20,400
gal.	152,500	152,500
Total volume		
cf	81,500	81,500
gal.	610,000	610,000
Number of tanks in service	4	4
Detention time, minutes	7	3
Overflow rates, gpsf/day	26,700	61,000
Air supply range, cfm/ft	1-5	1-5
Total air supply range required, cfm	240-1,200	240-1,200
Number of blowers	2	2
Unit capacity range, cfm	120-600	120-600
Grit handling, cf grit/MG	4	4
Grit removed, cf/day	511	1,175
Heavy-duty detritus tanks		
Flow, mgd	260	292
Detention time, minutes	1.65	1.47
Total volume required, cf	39,800	39,800
Number of tanks	2	2
Tank dimensions		
Diameter, ft	65	65
Water depth, ft	6	6
Unit area, sq ft	3,320	3,320
Total area, sq ft	6,640	6,640
Unit volume, cf	19,900	19,900
Unit volume, gal.	148,700	148,700
Total volume, cf	39,800	39,800
Total volume, gal.	297,400	297,400
Number of tanks in service	2	2
Detention time, minutes	1.65	1.47
Overflow rates, gpsf/day	39,200	44,000
Grit handling, cf/mil/gal.	4	4
Grit removed, cf/day	1,040	1,170

TABLE IV-7 (concluded)

HOLDING BASINS

	Design flow
Flow, mgd	2.6
Number of channels	2
Channel dimensions	
Length, ft	100
Width, ft	18
Water depth at peak flow, ft	6.7
Unit area, sq ft	1,800
Total area, sq ft	3,600
Unit volume, cf	12,100
Unit volume, gal.	90,200
Total volume, cf	24,200
Total volume, gal.	180,400
Detention time at peak flow, hr	1.6

OVERFLOW TREATMENT FACILITIES-AIR FLOTATION

	Design flows	
	Average	Peak
Flow, mgd	23	35
Number of tanks	3	3
Tank dimensions		
Length, ft	100	100
Width, ft	18	18
Depth, ft	8	8
Unit area, sq ft	1,800	1,800
Total area, sq ft	5,400	5,400
Unit volume		
cf	14,400	14,400
gal.	141,000	141,000
Total volume		
cf	43,200	43,200
gal.	323,000	323,000
Recycle rate, percent	33	33
Recycle pumps		
Number	3	3
Capacity, each, gpm	1,800	1,800
Detention time, minutes		
Hydraulic	20	13
Effective	26.7	17.3
Overflow rate, gpsf/day	4,320	6,480
Air supply, cfm	50	50
Number of blowers	2	2
Chlorine dose rate, mg/l	25	25

1 Accession Number	2 Subject Field & Group	
		SELECTED WATER RESOURCES ABSTRACTS **INPUT TRANSACTION FORM**

5 Organization

Metcalf & Eddy, Inc., Palo Alto, California

6 Title

STORM WATER PROBLEMS AND CONTROL IN SANITARY SEWERS,

10 Author(s)	11 Date	12 Pages	15 Contract Number
Pound, Charles E.			
	16 Project Number: FWQA Contract No.14-12-4[illegible] (11024 EQG)	21 Note	

22 Citation

Water Pollution Control Research Series - 11024 EQG 3/71

23 Descriptors (Starred First)

*Infiltration, *Sewerage, *Overflows, *Flow measurements, *Computer models, Sampling, Sewage treatment, Storm water

25 Identifiers (Starred First)

*Hydrology, Estimated costs

27 Abstract

An engineering investigation was conducted on storm water infiltration into sanitary sewers and associated problems in the East Bay Municipal Utility District, Special District No. 1, with assistance from the cities of Oakland and Berkeley, California. Rainfall and sewer flow data were obtained in selected study subareas that characterized the land use patterns predominant in the study area. Results obtained were extrapolated over larger drainage areas. A computerized flow routing program for the sewer system was used in this analysis. Ratios of infiltration to rainfall in the study subareas range from 0.01 to 0.14. Ratios of peak wet weather flow to average dry weather flow range from 2.1 to 9.1. About 11.1 percent of the rainfall enters the sanitary sewer system; 30.6 percent of the infiltration is contributed by the 4 percent of the study area that has combined sewers. Problems associated with infiltration and resulting overflows and bypasses are: 1) pollution of San Francisco Bay, 2) operational difficulties at the treatment plant, and 3) danger to public health, property damage, and nuisance. Estimated costs for the most feasible combinations of solutions to these problems, consisting of treatment plant improvements, separation of remaining combined sewers, partial treatment of overflows, and sewer improvements, range from approximately $42 million to $94 million. Specific recommendations for subsequent developmental programs are presented; complete implementation of the recommended plan will take about 7 years.

Abstractor: Charles E. Pound

Institution: Metcalf & Eddy, Inc.

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Continued from inside front cover....

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11023 --- 09/67	Demonstrate Feasibility of the Use of Ultrasonic Filtration in Treating the Overflows from Combined and/or Storm Sewers
11020 --- 12/67	Problems of Combined Sewer Facilities and Overflows, 1967 (WP-20-11)
11023 --- 05/68	Feasibility of a Stabilization-Retention Basin in Lake Erie at Cleveland, Ohio
11031 --- 08/68	The Beneficial Use of Storm Water
11030 DNS 01/69	Water Pollution Aspects of Urban Runoff, (WP-20-15)
11020 DIH 06/69	Improved Sealants for Infiltration Control, (WP-20-18)
11020 DES 06/69	Selected Urban Storm Water Runoff Abstracts, (WP-20-21)
11020 --- 06/69	Sewer Infiltration Reduction by Zone Pumping, (DAST-9)
11020 EXV 07/69	Strainer/Filter Treatment of Combined Sewer Overflows, (WP-20-16)
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11020 FKI 01/70	Dissolved Air Flotation Treatment of Combined Sewer Overflows, (WP-20-17)
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11023 FDD 03/70	Rotary Vibratory Fine Screening of Combined Sewer Overflows, (DAST-5)
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